AD-A149 772 MURRELLS INLET NAVIGATION PROJECT GEORGETOWN COUNTY SOUTH CAROLINA REVISI..(U) CORPS OF ENGINEERS CHARLESTON SC CHARLESTON DISTRICT SEP 76 1/1

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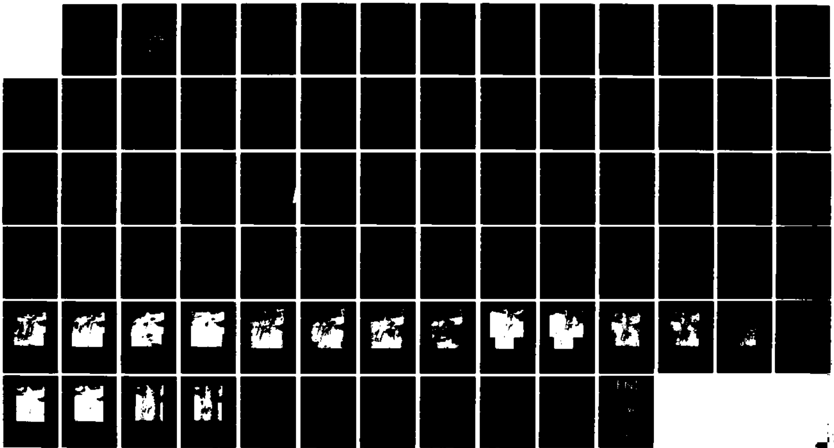

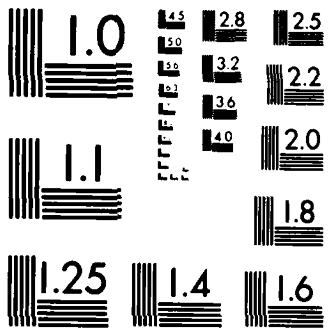
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AD-A149 772

SUPPLEMENT #1

DESIGN MEMORANDUM 1
GENERAL DESIGN

MURRELLS INLET NAVIGATION PROJECT
GEORGETOWN COUNTY, SOUTH CAROLINA

REVISION OF WEIR SYSTEM AND JETTY & CHANNEL ALIGNMENTS

U.S. ARMY ENGINEER DISTRICT, CHARLESTON

CORPS OF ENGINEERS

Charleston, South Carolina

Sept. 1976

COPY NO. 12

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DEPARTMENT OF THE ARMY

CHARLESTON DISTRICT, CORPS OF ENGINEERS
P O BOX 919
CHARLESTON, S.C. 29402

SANGE-D 14 September 1976

SUBJECT: Murrells Inlet, South Carolina - Supplement No. 1 to the General Design Memorandum - Revision of Weir System and Jetty and Channel Alignments

Division Engineer, South Atlantic
ATTN: SADEN-GK

1. Transmitted are 18 copies of Supplement No. 1 to the General Design Memorandum, submitted for approval in accordance with applicable provisions of ER 1110-2-1150, dated 1 October 1971, as revised 22 July 1974 by change 7, SAD Supplement 1 to ER 1110-2-1150 and DvR 1110-1-5, dated 4 April 1973.

2. It is recommended that this supplement be approved as the basis for preparation of plans and specifications.

3. As a result of recent congressional action, substantial funds have been added to the current appropriations bill in order to initiate construction of the Murrells Inlet Project in mid-FY 77. Therefore, it is requested that the revised plan receive a timely review to expedite preparation of contract plans and specifications.

1 Incl (18 cys)
as

HARRY S. WILSON, JR.
Colonel, Corps of Engineers
District Engineer

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MURRELLS INLET NAVIGATION PROJECT
GEORGETOWN COUNTY, SOUTH CAROLINA

Supplement No. 1 to General
Design Memorandum

PERTINENT DATA

DESIGN DETAILS

1. North Jetty

Total Length of Jetty (Excl. Sand Dike)	3,455'
Type of Construction	Quarrystone
Jetty Head:	
Length	150'
Crest Elevation	+9' MLW
Crest Width	18'
Side Slopes	1V on 2H
Armor Stone I Size	6-10 tons
Jetty Trunk (seaward)	
Length	1,355'
Crest Elevation	+9' MLW
Crest Width	15'
Side Slopes	1V on 2H
Armor Stone II Size	4-7 tons
Cover Stone (Weir Section)	
Effective Length	1,315'
Crest Elevation (Average)	+2.2 MLW
Crest Width (maximum)	15'
Side Slopes	1V on 2H
Cover Stone Size	0.56-2.50 tons
Jetty Trunk (Landward)	
Length	620'
Crest Elevation	+9.0 MLW
Crest Width	15'
Side Slopes	1V on 2H
Armor Stone II Size	4-7 tons

2. <u>Deflector Dike</u>

Length	1,300'
Crest Elevation	Varies
Crest Width	5'
Side Slopes	1V on 2H
Rubble Stone Size	100-900 lbs

3. <u>North Sand Dike</u>

Length	500'±
Crest Elevation	+10' MLW
Crest Width	100'
Side Slopes	1V on 10H

4. <u>South Jetty</u>

Total Length of Jetty (Excl. Sand Dike)	3,330'
Type of Construction	Quarrystone
Jetty Head:	
Length	150'
Crest Elevation	+9' MLW
Crest Width	18'
Side Slopes	1V on 2H
Armor Stone I Size	6-10 tons
Jetty Trunk:	
Length	3,180'
Crest Elevation	+9' MLW
Crest Width	15'
Side Slopes	1V on 2H
Armor Stone II Size	4-7 tons

5. <u>South Sand Dike</u>

Length	2,850'±
Crest Elevation	+10' MLW
Crest Width	100'
Side Slopes	1V on 25H

6. Navigation Channels

	Entrance	Inner Channel	Inner Channel A	Inner Channel B
Length	3,000'	15,440	1,850'	13,590'
Bottom Width	300'	-	200'	90'
Project Depth	-10' MLW	-	-10' MLW	-8' MLW
Allowable Overdepth	2'	-	2'	2'
Side Slopes	1V on 4H	1V on 4H	1V on 4H	1V on 4H

7. Auxiliary Channel (To Oaks Creek)

Length	670'
Bottom Width	200'
Depth	-10' MLW
Allowable Overdepth	2'
Side Slopes	1V on 4H

8. Deposition Basin

Dimensions	100' X 930' X 570' X 660' X 1,300
Depth	-18' MLW
Allowable Overdepth	2'
Side Slopes	1V on 4H
Capacity	600,000 Cu. Yds.

9. Estimate of Project First Costs

01.	Lands and Damages	$1,050,000
09.	Channels	$1,971,000
10.	Jetties	$9,788,000
14.	Recreation Facilities	$ 286,000
30.	Engineering & Design	$1,018,000
31.	Supervision & Administration	$ 602,000
	Total Project First Cost	$14,715,000

10. Annual Economic Charges - Total

Total Project	$1,432,000
Navigation Project	$1,403,000

11. <u>Annual Benefits</u>

Navigation	2,015,000
Recreation	36,000
Redevelopment	93,000
Total	$2,144,000

12. <u>Benefit-Cost-Ratio</u>

BCR (total)	1.50
BCR (navigation only)	1.44

Murrells Inlet Navigation Project
Georgetown County, South Carolina

Supplement No. 1

Design Memorandum 1

General Design

INTRODUCTION

1. Authorization. The Murrells Inlet Navigation Project, as presented in House Document No. 92-137, was approved by House Resolution of the Public Works Committee, dated 10 November 1971, and by similar Senate Resolution of the Public Works Committee, dated 18 November 1971.

2. The preparation of this supplement was required by paragraph 4 of SADPD-P (2 Dec 75) 1st Indorsement dated 20 April 1976, subject: Murrells Inlet, South Carolina Design Memorandum 1 - General Design Memorandum; and SADEN-GK (19 Mar 76) 1st Indorsement dated 25 March 1976, subject: Design Memorandum for Murrells Inlet, South Carolina.

3. Purpose. This supplement presents modifications of the channels, jetties and weir system that were previously submitted for approval. The major revisions occurred as a result of model test data developed by WES subsequent to submission of the GDM. Other changes were made as a result of a conference held at SAD on 20 August 1976 concerning design of weir jetty structures.

4. Scope. This supplement covers changes in jetty and channel configuration resulting from model testing of Plans 7A, 7B, Plans 1B through 1H and from innovative changes in weir system design. It also presents a summary of the WES model results for all plans tested. A comprehensive report of the model testing program for the Murrells Inlet Navigation Project will be submitted as a separate appendix (to the General Design Memorandum) at a later date. A revised project cost estimate considering the various changes is presented.

MODEL STUDY

5. General. A physical model of Murrells Inlet and estuary was constructed at WES to evaluate the effects of currents and wave action on different arrangements of the jetty system under simulated prototype conditions. This fixed bed model was constructed to 1:200 horizontal and 1:60 vertical scales.

6. Previous plans. Originally, WES proposed seven jetty alignments for preliminary testing (Plans 1-7). From these alignments, Charleston District selected Plans 1, 2, 4, 6 and 7. These plans are shown on Figures 1 through 8. A discussion of the plans selected for preliminary testing follows:

a. Plan 1. This alignment (Figure 1) was essentially the same as for the project plan presented in the survey report, but with the following changes: the deposition basin was made larger and an access channel (cut between the basin and the intersection of the entrance and inner channels) was provided. The deposition basin was empty for the tests. Testing revealed that the ebb and flood flows of Oaks Creek were blocked. Flows into and out of Oaks Creek became very circuitous, and caused surface currents to be stronger toward the south side of the entrance channel. Such a condition presented a threat of southward migration of this channel and eventual scouring of the south jetty and sand dike.

b. Plan 1A. This plan (Figure 2) was a variation of Plan 1 containing a modification providing for a 300-foot wide connecting channel between Oaks Creek and the entrance channel. Surface current photographs showed that this auxiliary channel helped the flow in and out of Oaks Creek and lessened the possibility of scour at the south jetty and sand dike. WES felt that the auxiliary channel could be narrowed to increase the velocity and enhance flushing action.

c. Plan 2. This scheme was basically Plan 1 with no low weir section on the north jetty (see Figure 3). Photographs showed that Oaks Creek flow impinges on the left side of the entrance channel. Surface current photography also produced evidence that a shoal may develop at the ends of the jetties and between Oaks Creek and Woodland Creek. The shoal between the two creeks formed due to low velocities; however, it did not develop in testing of Plans 1 and 1A.

d. Plan 4. This plan (Figure 5) reoriented the jetties such that they were more normal to the existing coastline. The north and south jetty were constructed of equal length. This plan also includes an auxiliary channel connecting the entrance channel to Oaks Creek. High velocities were evidenced in the inner channel which could cause navigation problems for smaller boats. Surface current photographs show a problem with flows around the ends of the jetties; however, this alignment would probably cause less scour than Plans 1, 1A and 2.

e. Plan 6. Plan 6 was the same as Plan 4 but without a low weir section. Flow through the jetties appeared to be more centered which is seemingly caused by absence of the weir section. This plan is shown on Figure 7.

f. Plan 7. This alignment (Figure 8) was similar to Plans 1, 1A and 2 except the jetty system was shifted toward the south (closer to Huntington Beach). This configuration better utilized the existing channel through the inlet. The north and south jetties were equal in length and longer than for Plans 1, 1A and 2. This system was aligned such that a connection to Oaks Creek was provided without dredging a special channel. Surface current photographs showed flows around the south jetty end that could cause scour problems.

7. Full scale testing. On 19 June 1975 in Charleston, WES, District and SAD representatives met to discuss the tested plans. As a result of these discussions it was decided that full scale model tests should be conducted on Plans 1A and 7. However, Plan 1A was modified to include certain changes and designated Plan 1B as discussed below.

8. Plan 1B. This alignment (Figure 9) was a variation of Plan 1A. The auxiliary channel from Oaks Creek to the entrance channel was reduced from 300 feet to 200 feet wide to provide greater velocities and the south jetty extended to be equal in length to the north jetty to improve generally the hydraulic conditions at the jetty entrance.

9. GDM Plan. As a result of discussions with WES and higher authority considering model testing information through October 1975, Plan 1B was selected as the basic scheme for presentation in the GDM. However, as a result of informal review comments on the GDM from SAD, dated 19 February 1976, the proposed GDM plan was changed from Plan 1B to include reductions in the entrance channel depth from 12 to 10 feet and inner channel depth from 10 to 8 feet. Time precluded testing of these modifications prior to GDM submittal. However, in discussions with WES, it was felt that the modifications would not significantly effect the results of Plan 1B in critical areas of concern.

10. Additional Testing. During the period immediately following submittal of the GDM, model activity was confined to minor modification of Plan 1B to Plan 1C and evaluation of testing of Plans 7A and 7B on a comparative feature basis with Plans 1B and 1C. Testing of channel depth changes reflected in the GDM plan were initiated with the testing of Plan 1D. A discussion of additional tests follows:

a. Plan 1C. This plan (Figure 10) was a variation of Plan 1B to provide an increased width of from 200 to 300 feet in the auxiliary channel from Oaks Creek to the entrance channel. This was done in an effort to reduce excessively high ebb velocities in the auxiliary channel during testing of Plan 1B. However, this modification produced only a slight reduction of the ebb velocities.

b. Plans 7A, 7B. Plan 7 was scheduled for testing after Plan 1C. To better evaluate the effects of velocity in the auxiliary channel, Plan 7A and Plan 7B were developed. Plan 7A (Figure 11) was constructed with a 200-foot wide auxiliary channel having an invert elevation of -6.0 feet mlw. A 300-foot wide auxiliary channel was proposed for Plan 7B (Figure 12).

c. A meeting of WES, SAD, OCE and District representatives was held at WES on 15 and 16 March 1976 to consider the progress and results of the full scale testing program. Tests showed Plans 7A and 7B produced less favorable results than Plans 1B and 1C. After considering this along with time, costs and benefits of further testing on Plans 7A

and 7B, it was agreed that WES should confine subsequent testing to optimization of the Plan 1 scheme continuing with consideration of Plan 1C results and the GDM plan.

d. Plan 1D. As a result of testing and evaluation of Plan 1C and in recognition of the channel depth changes in the GDM plan, the following major revisions were made to develop Plan 1D (Figure 13):

(1) The jetty spacing was reduced from 900 feet to 600 feet wide to enhance the flushing action by increasing the velocity in the entrance channel.

(2) The proposed entrance channel was reduced in depth from -12 feet mlw to -10 feet mlw in order to increase the ebb velocities so that the probability of channel shoaling would be reduced.

(3) The depth of the proposed auxiliary channel leading to Oaks Creek was increased from -6 feet mlw to -10 feet mlw and the width was established at 200 feet to reduce the high velocities that could cause scouring and navigation hazards to smaller boats.

(4) The width of the initial section (Inner Channel A) of the proposed inner channel was increased from 90 to 200 feet; and, the depth was increased from -8 feet mlw to -10 feet mlw.

e. Surface current photography showed a pronounced tendency for ebb currents in this plan to migrate toward the north causing concern of a threat to the deposition basin and the north jetty.

f. Plan 1E. Plan 1E was developed in an attempt to alleviate concerns about the ebb current that arose during testing of Plan 1D. This scheme (Figure 14) extended the north jetty 500 feet landward (parallel to the entrance channel) and moved the deposition basin access channel more seaward. Unfortunately, Plan 1E did not effectively eliminate the potential current migration problem.

g. Plan 1F. Because of undesirable results with the previous configuration, Plan 1F was developed. This scheme eliminated the 500-foot jetty extension; and, was merely the same as Plan 1D but with the areas around the deposition basin and the weir filled to -2 feet mlw. Plan 1F is shown on Figure 15. Surface current photographs showed the same pronounced tendency of current migration as did Plan 1D.

h. Plan 1G. This plan (Figure 16) was essentially the same as Plan 1D but with a 1,300 foot deflector dike added to extend from the Garden City peninsula around the north side of the deposition basin. Its crest elevation remained constant at +9.0 feet mlw along its entire length. Surface current photography showed that the ebb currents were effectively deflected and remained within the dredged channels.

i. Plan 1H. Plan 1H (Figure 16a) was developed to improve hydraulic characteristics through the weir and deposition basin during flood flows. This plan is identical to Plan 1G except that the training dike has a crest of varying elevation. Starting at the dune line, the crest varied from +9.0 feet mlw to +2.3 feet mlw approximately 600 feet from the dune line; thence, remained constant at +2.3 feet mlw to the end of the dike. This plan produced the most favorable results of all the previous model testing.

11. GDM Supplement Plan. At WES' recommendation, Plan 1H has been adopted as the project plan for presentation in this GDM Supplement. However, the District proposes not to construct the deflector dike as a part of initial construction. The reasons for this decision are as follows:

(1) The dike is considered to be an extreme safety hazard to navigation of small boats. Upon entering the inlet from the ocean through the jettied opening, a boat operator would not expect another rock structure to be projecting into the open water and the chance of serious accidents, particularly during periods of darkness or inclement weather, would be very great. A boat operator not following the navigation channel (and many small boat operators do not) may run aground on a sand bar but this would not have any serious effect on his personnal safety. The proposed deflector dike would constitute a serious safety hazard and should be proven to be required based on prototype conditions prior to serious consideration of its construction.

(2) The hydrography of the inlet has changed considerably since the model configuration was moulded. Pipeline dredging operations have recently been completed to straighten and deepen the inner channel between the south end of Garden City and the marshland to the west. The new channel is located further northwest (toward the marsh), and therefore further from the southern tip of Garden City than the previous channel. There is considerable ebb flow in the newly dredged channel and there is reason to believe that the water area northwest of the tip of Garden City will shoal significantly due to lower ebb velocities in this area. The hydrography in the weir and deposition basin area has changed (shallowed) significantly since the model was constructed and prototype conditions may well not produce the migrating ebb currents that the model tends to indicate. Prototype conditions may well exist at the time of construction which would obviate the need for a deflector dike of any kind.

12. The District recommends that the project be built without the deflector dike and that the currents be monitored during and following construction to determine if a need exists for a deflector to protect the deposition basin. If determined to be required, a deflector could be designed to suit the then existing conditions and constructed in a short period of time, well in advance of any real channel migrating problems. Therefore, a deflector dike is included as a contingency item, subject to prototype investigations; however, its cost is included as an item of first cost.

13. Surface current photographs for Plans 1B, 1C, 7A, 7B, 1D, E, F, G and H are shown on Figures 17 through 34.

14. Hurricane tests. The GDM proposed that hurricane tests be conducted on the final project plan. However, on 27 April 1976 (telecon), SAD and OCE informed Charleston District that the hurricane tests would not be required.

15. Channels. Channel widths and depths were changed as a result of model testing. See subparagraphs 10d (2), (3) and (4) above for details.

16. Jetties. Jetty spacing was reduced as a result of model testing. See subparagraph 10d (1), above for details. A jetty trunk section has been added to facilitate transitioning between the stone weir and the high land on the Garden City peninsula.

17. Weir. The weir section of the north jetty system has been changed from a concrete sheet pile type proposed in the GDM to a stone type. This change resulted from a conference at SAD on 20 August 1976 attended by representatives from CERC, OCE, SAD, and interested Districts of SAD. The stone weir system would be constructed to function in the same manner as the previously proposed concrete pile type. In effect, the weir crest would be established low enough to allow longshore drift to bypass into the deposition basin, but high enough to protect a dredge operating in the deposition basin under reasonably stable weather conditions.

Materials handling and weir construction would be accomplished with the same equipment and procedures described in the GDM for jetty construction. The weir section would be constructed starting from the landward end. In order to minimize scour during armor stone placement, the contractor would be required to maintain the foundation blanket a minimum of 200 feet ahead of the remaining weir construction.

Structurally, the stone weir system would have characteristics similar to the stone jetties. This system would be constructed of a foundation blanket, toe protection and armor stone (see Plates 2 and 3). The newly designed weir system is considered justified because of the following advantages over the concrete sheet pile weir.

a. The stone weir cost less to construct.

b. Quality control of the stone system would be less critical to maintain during construction.

c. The stone weir would require no additional specialized equipment at the site.

d. After construction, the stone weir section could be adjusted and repaired with less difficulty by merely "adding on" or "taking off" armor stone.

18. Deflector Dike. As a result of model testing it may be necessary to construct a deflector dike extending from the Garden City peninsula around the upper side of the deposition basin in order to prevent channel migration toward the deposition basin and north jetty. The deflector dike is shown on Plates 1, 1A and 3 and discussed in detail in paragraph 29. Model study information is presented in subparagraph 10i.

Project Description

19. General. The proposed plan in this supplement (designated Plan 1H) provides for the construction of a north jetty with a low weir section, a south jetty, sand dikes, a deposition basin, entrance, inner and auxiliary channels, and recreation facilities. The proposed plan has the same general characteristics as the GDM Plan. However, the proposed plan incorporates some significant design improvements as discussed in paragraphs 10 and 15 through 18, above. These changes are shown on Plates 1, 1A, 2 and 3.

20. North jetty. The proposed north jetty and weir system would be constructed entirely of quarrystone from the shoreward end of an existing dune line to the -10 feet mlw ocean contour. The jetty would consist of a head section, a low weir section and two trunk sections as shown on Plate 2. The jetty section would start at a sand dune with a crest elevation of +9.0 feet mlw and continue for a distance of 518 feet; then transition from +9.0 feet mlw to +2.2 feet mlw (with a 1V on 15H slope) to the low weir section.

21. The low weir section would allow the passage of littoral drift traveling essentially between the shoreline and the -4 foot ocean contour. The effective weir section would be 1,315 feet long and have an average crest elevation of +2.2 feet mlw. The jetty trunk would then transition from +2.2 feet mlw to +9.0 feet mlw with a 1V on 2H slope. The jetty would remain at +9.0 feet mlw to its end (approximately 1505 feet). The total length of the north jetty is 3,455 feet. The head section consists of the outer 150 feet of jetty.

22. The head section would have two armor layers of 6-10 ton stones, a maximum crest width of 18 feet and side slopes of 1V on 2H. The jetty trunk - ocean side of weir - from the head to the -6 feet mlw ocean contour would have two armor layers of 4-7 ton stones, a maximum crest width of 15 feet and side slopes of 1V on 2H. The jetty trunk (from the -6 feet mlw contour to the weir) would have a single armor layer of 4-7 ton stones, a maximum crest width of 15 feet and side slopes of 1V on 2H. The weir section would have a single armor layer of variable size stones (ranging from 900 lbs. to 2.5 tons), a maximum crest width of 15 feet and side slopes of 1V on 2H. These stones would range from 2 feet to 3.5 feet in diameter and would provide a single cover layer along the weir. The landward jetty trunk would have a single armor layer of 4-7 ton stones, a maximum crest width of 15 feet and side slopes of 1V on 2H.

23. South jetty. The proposed south jetty would be constructed from a new sand dike (terminating at the -2 feet mlw contour) to the -10 feet mlw ocean contour. The jetty would be constructed entirely of quarrystone for a distance of 3,330 feet. The top elevation of the jetty stones would be +9 feet mlw; the top of the fishing walkway would be +10 feet mlw. The jetty would consist of three sections: a head section and two trunk sections, constructed in the same manner as described for the north jetty on the ocean side of weir structure.

24. Sand dikes. The south sand dike would be constructed from the shoreward end of the stone jetty to the existing dune line at +10 feet mlw elevation. The north sand dike would be constructed from the landward trunk section to the existing dune line at +10 feet mlw elevation. The sand dikes would connect the jetties to the existing high ground. The south sand dike would extend from an existing dune line to -2 feet (mlw) ocean contour, a length of about 2,850 feet. The north sand dike would consist of strengthening (by widening) an existing sand dune for a distance of about 500 feet. The dikes would have a crest width of 100 feet. The slopes for the north dike would be 1V on 10H; the slopes for the south dike would be 1V on 25H. The dikes would be constructed by hydraulically placed granular fill dredged from the proposed channels and deposition basin. Upon completion of construction, the sand dikes would be planted with sea oats or other salt-tolerant plant species to aid in erosion control.

25. Deposition basin. Following construction of the jetties, a deposition basin would be dredged with a pipeline dredge between the north jetty and northern limit of the entrance channel to trap littoral material moving southward over the weir section. The basin would be dredged to a depth of -18 feet mlw and would have a capacity of 600,000 cubic yards. An allowable overdepth of 2 feet would be permitted to compensate for dredging inaccuracies. The side of the basin adjacent to the weir would be 1,300 feet long; the other dimensions are commensurate with the required basin capacity. The capacity of the deposition basin would be large enough to contain a three year accumulation of the estimated southward littoral drift (200,000 cubic yards per year).

26. Entrance channel. The entrance channel would extend from the -10 feet ocean contour to a point within the jetties, a length of 3,000 feet. The entrance channel would be 300 feet wide and 10 feet deep. An allowable overdepth of 2 feet would be permitted to compensate for dredging inaccuracies. An additional overdepth of 2 feet to facilitate future maintenance in areas of hard bottom material would not be required. Since beach sands are known to compact very hard due to the vibratory action of the surf, it is believed that any shoal material (littoral drift) would compact just as hard. The compaction of the shoal material to the same degree as the in situ material would negate any possible benefits from advance maintenance overdepth. Side slopes of 1V on 4H would be expected initially after the box-cut dredging of the channel. Due to the wave action in the entrance channels, the ultimate side slope would probably be 1V on 10H. The distance between the edge of the channel and the jetty toe would be sufficient to allow an ultimate side slope of 1V on 10H, and at the same time provide a minimum distance of 25 feet to the toe of either jetty system.

27. Inner channel. The inner channel (consisting of Inner Channel A and Inner Channel B) would extend from the entrance channel through Main Creek to the old Army crash boat dock, a length of 15,440 feet, where it would terminate with a turning basin 300 feet long and 150 feet wide. Inner Channel A - starting at the entrance channel and extending 1,850

feet - would be 200 feet wide and have a bottom elevation of -10 feet mlw. Inner Channel B would be 13,590 feet long, 90 feet wide and would have a bottom elevation of -8 feet mlw. An allowable overdepth of 2 feet would be permitted to compensate for dredging inaccuracies. An additional overdepth of 2 feet to facilitate future maintenance in areas of hard bottom material would not be required. Side slopes of 1V on 4H would be expected after the box-cut dredging of the channel. Since there is little or no wave action in the inner channel, it is believed that this slope would remain stable once dredged.

28. Auxiliary channel. The auxiliary channel would extend from the entrance channel to the -10 foot contour at the mouth of Oaks Creek, a length of 670 feet. The auxiliary channel would be 200 feet wide and 10 feet deep. An allowable overdepth of 2 feet would be permitted to compensate for dredging inaccuracies. This channel would be dredged initially (only); there would be no annual maintenance.

29. Deflector dike. The deflector dike would extend from an existing dune line (at elevation +9.0 feet mlw) on the Garden City side of the project into the inlet for a distance of 1,300 feet. The crest elevation would continually slope from +9.0 feet mlw to +2.3 feet mlw as shown on Plate 3; thence, remain constant at elevation +2.3 feet mlw to the end of the dike. It would be constructed of 100-900 pound stones and have no prepared foundation. The deflector dike would have a maximum crest width of 5 feet and side slopes of 1V on 2H. The purpose of the dike would be to deflect any ebb flow tending to migrate through the deposition basin. The deflector dike would not be planned for initial construction, but would be built at a later date should migration become evident.

30. Disposal area. A 16± acre disposal area would be located on highland for the disposal of dredged material unsuitable for placement on the beach (sand with high silt or clay content). A 4± acre disposal area would be located on the beach front for the disposal (during initial construction only) of dredged material suitable for placement on the beach. During construction, excess dredge material suitable for beach placement would be deposited in areas designated on the drawings as nourishment areas. After construction, suitable material would be placed in the surf zone or on adjacent beaches where necessary as part of the sand-bypassing operation.

31. Recreation facilities. An 8-foot wide fishing walkway of asphaltic concrete would be located on the crest of the south jetty. The walkway would extend from the sand dike to the jetty head, for a length of about 3,330 feet. A parking area for 100 vehicles would be located adjacent to an existing parking area at Huntington Beach State Park. A comfort station would also be provided adjacent to the existing parking area. A complete discussion of the proposed recreation facilities is contained in the main report.

Cost Estimates

32. Cost estimates. Estimated cost of Murrells Inlet Navigation Project was determined using quantity estimates derived from field surveys, land appraisals, and foundation investigations. Cost estimates are based on past experience and October 1976 contract prices applied to the estimated quantities. Costs covering contingencies, engineering and design, and supervision and administration are included in the estimates. A summary cost estimate of project first cost is presented in Table 1. A detailed cost estimate of Murrells Inlet is given in Table 2.

33. Comparison with prior estimates. A comparison between the current estimate (price levels October 1976) reflected in this report and the latest approved PB-3 estimate (effective 1 October 1976), is presented in Table 3. The GDM estimate at October 1975 price levels is shown in this table. The total overall cost of the project as presented in this supplement has decreased approximately $85,000 below the approved PB-3 estimate (effective October 1976). This overall reduction in project cost is due to the following:

(1) Increase of $167,000 in Lands and Damages due to additional land requirements (on the Garden City Peninsula) for construction of the Deflector Dike.

(2) Decrease of $313,000 in Channels and Canals due to less excavation quantities resulting from a more favorable hydrography (determined from later surveys) and from a slight reduction in unit prices for dredging.

(3) Net increase of $42,000 in Breakwaters and Seawalls due to addition of the deflector dike and redesign of weir as a stone section in lieu of a concrete sheet pile type.

(4) Increase of $11,000 in Recreation Facilities due to lengthening of fishing walkway.

(5) Increase of $3,000 and $5,000 in accounts 30 and 31, respectively, due to refinement of the estimates.

(6) Increase of $313,300 in Non-Federal Costs due primarily to additional land requirements, and inadvertent use of 6.1% in the GDM to determine the local share of the Navigation Project. The local participation is actually 6.4%.

TABLE 1

SUPPLEMENT NO. 1 TO MURRELLS INLET GDM

SUMMARY PROJECT COST ESTIMATE

(October 1976 Price Levels)

Cost Account Number	Items or Feature	Current Cost Estimate
01.	Lands and Damages	$ 1,050,000
09.	Channels and Canals	1,971,000
10.	Breakwaters and Seawalls	9,788,000
14.	Recreation Facilities	286,000
30.	Engineering and Design	1,018,000
31.	Supervision and Administration	602,000
	TOTAL PROJECT COST	$14,715,000

TABLE 2

SUPPLEMENT NO. 1 TO MURRELLS INLET GDM
COST ESTIMATES

(October 1976 Price Level)

Cost Account	Feature	Unit	Quantity	Unit Cost	Total Cost
01.	LANDS AND DAMAGES				
	Fee Title				
	North Jetty and Sand Dike	L.S.	Job		$ 530,000
	Easements				
	Highland Disposal Area	L.S.	Job		220,000
	Highland Pipeline	L.S.	Job		40,000
	Drainage Ditch	L.S.	Job		5,000
	Beach Disposal	L.S.	Job		65,000
	Pipeline, Bypass	L.S.	Job		35,000
	North Construction Area	L.S.	Job		20,000
	Subtotal				$ 915,000
	Contingencies				135,000
	Account 01. Total				$1,050,000
09.	CHANNELS AND CANALS				
	Mobilization and Demobilization	L.S.	Job		150,000
	Excavation, Unclassified:				
	Inner Channel	C.Y.	200,000	$1.00	200,000
	Auxiliary Channel	C.Y.	64,000	1.00	64,000
	Entrance Channel	C.Y.	320,000	1.10	352,000
	Deposition Basin	C.Y.	600,000	1.30	780,000
	Disposal Area Preparation	L.S.	Job		38,000
	Aids to Navigation	L.S.	Job		130,000
	Subtotal				$1,714,000
	Contingencies, 15%				257,000
	Account 09. Total				$1,971,000

TABLE 2
(cont.)
SUPPLEMENT NO. 1 TO MURRELLS INLET GDM
COST ESTIMATES

(October 1976 Price Level)

Cost Account	Feature	Unit	Quantity	Unit Cost	Total Cost
10.	BREAKWATERS AND SEAWALLS				
	.1 North Jetty				
	Armor Stone I (6-10 ton)	Ton	9,000	$35.00	$ 315,000
	Armor Stone II (4-7 ton)	Ton	48,200	33.00	1,591,000
	Cover (weir) Stone	Ton	2,400	33.00	79,000
	Core Stone	Ton	26,000	30.00	780,000
	Foundation Blanket	Ton	24,700	31.00	766,000
	Excavation	CY	5,000	4.00	20,000
	Account 10.1 Subtotal				$3,551,000
	.2 South Jetty				
	Armor Stone I (6-10 ton)	Ton	8,200	$35.00	$ 287,000
	Armor Stone II (4-7 ton)	Ton	73,400	33.00	2,422,000
	Core Stone	Ton	32,600	30.00	978,000
	Foundation Blanket	Ton	35,400	31.00	1,097,000
	Account 10.2 Subtotal				$4,784,000
	.3 Deflector Dike				
	Rubble Stone	Ton	3,300	$30.00	$99,000
	.4 Sand Dikes Erosion Control	L.S.	Job		$77,000
	Account 10. Subtotal				$8,511,000
	Contingencies, 15%				1,277,000
	Account 10. Total				$9,788,000
14.	RECREATION FACILITIES				
	Fishing Walkway	L.F.	3,270	$55.00	$180,000
	Comfort Station	L.S.	Job		44,000
	Parking Lot	S.Y.	3,900	6.50	25,000
	Subtotal				$249,000
	Contingencies, 15%				37,000
	Account 14. Total				$286,000

TABLE 2
(cont.)
SUPPLEMENT NO. 1 TO MURRELLS INLET GDM
COST ESTIMATES

(October 1976 Price Level)

Cost Account	Feature	Unit	Quantity	Unit Cost	Total Cost
	Subtotal (Items 09., 10. and 14.)				$12,045,000
30.	ENGINEERING AND DESIGN (5%)				602,000
	Model Study				416,000
31.	SUPERVISION AND ADMINISTRATION (5%)				602,000
	TOTAL PROJECT COST				$14,715,000

R1 28 Dec 76

TABLE 3

COMPARATIVE ESTIMATE WITH PREVIOUS ESTIMATES
MURRELLS INLET, SOUTH CAROLINA
(ALL COSTS IN $1,000)
(FEDERAL AND NON-FEDERAL)

Item No.	Item	GDM Estimate (Oct 1975)	Approved Estimate (Effective Date 1 Oct 1976)	Current Estimate (Oct 1976)
01.	Lands and Damages	$ 815	$ 883	$ 1,050
09.	Channels and Canals	2,075	2,284	1,971
10.	Breakwaters and Seawalls	9,153	9,746	9,788
14.	Recreation Facilities	259	275	286
30.	Engineering and Design	990	1,015	1,018
31.	Supervision & Administration	574	597	602
		$13,866	$14,800	$14,715

Current Estimate (Oct 1975)

NON-FEDERAL COSTS (Cash Contribution and Land and Damages)
$2 082,000

(Reimbursement for 6.4% of cost of Navigation Facilities, 50% of Cost of Recreation Facilities and All Lands Required)

(1) Includes Cost of Aids to Navigation

BENEFIT-COST-RATIO

34. Benefit-Cost-Ratio. The benefit-cost-ratio (BCR) was revised due to adoption of Plan 1H as the final project plan. The BCR resulting from tangible navigation benefits (only) is revised as follows:

Annual Benefits	Annual Costs	BCR	Excess of Benefits Over Costs
$2,015,000	1,403,000	1.44	612,000

RECOMMENDATIONS

35. Recommendations. It is recommended that the proposed plan of improvement described in this supplement be approved as a basis for development of final design plans and specifications for eventual construction of the Murrells Inlet Navigation Project.

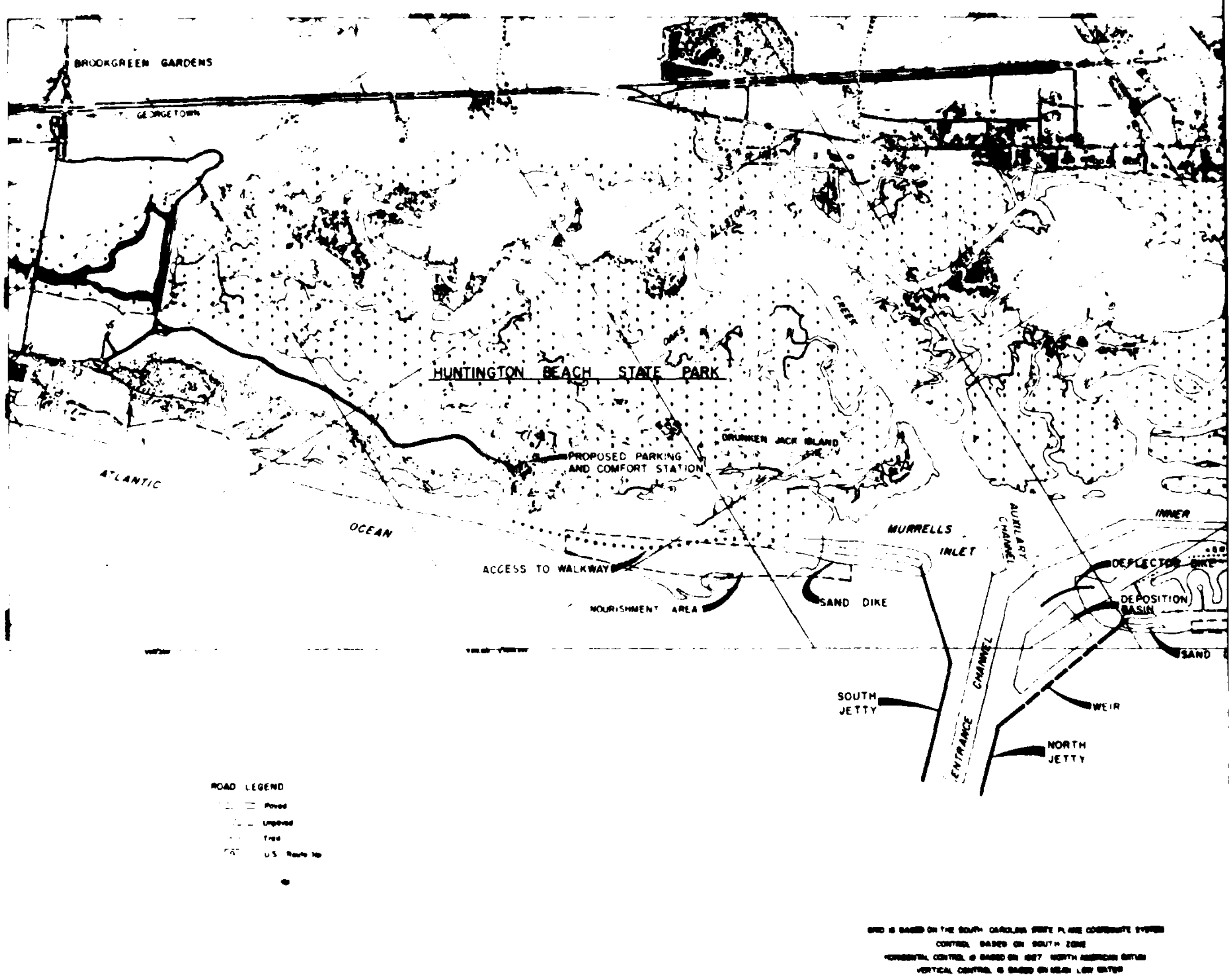
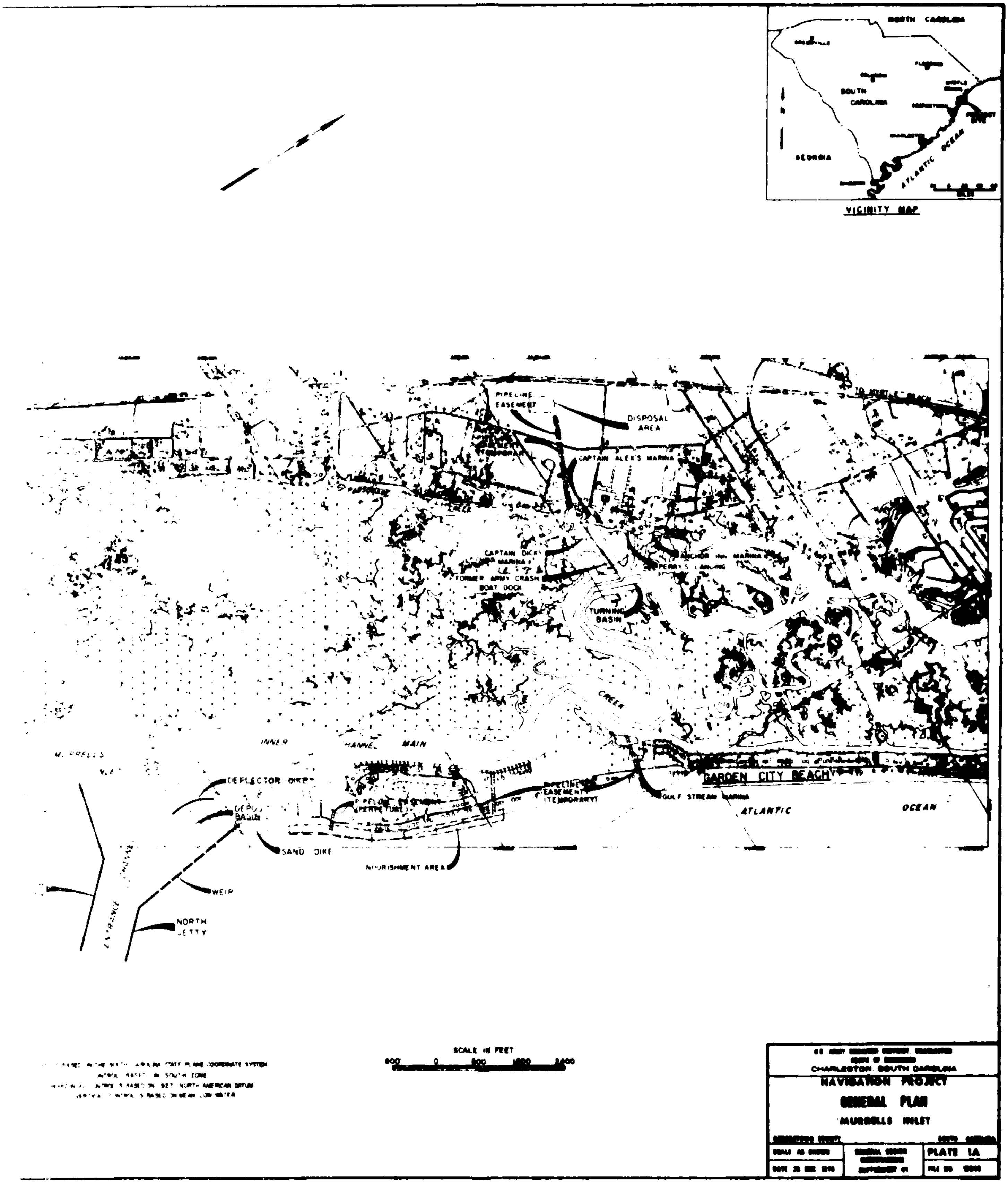


PLATE 1A

1 of 2



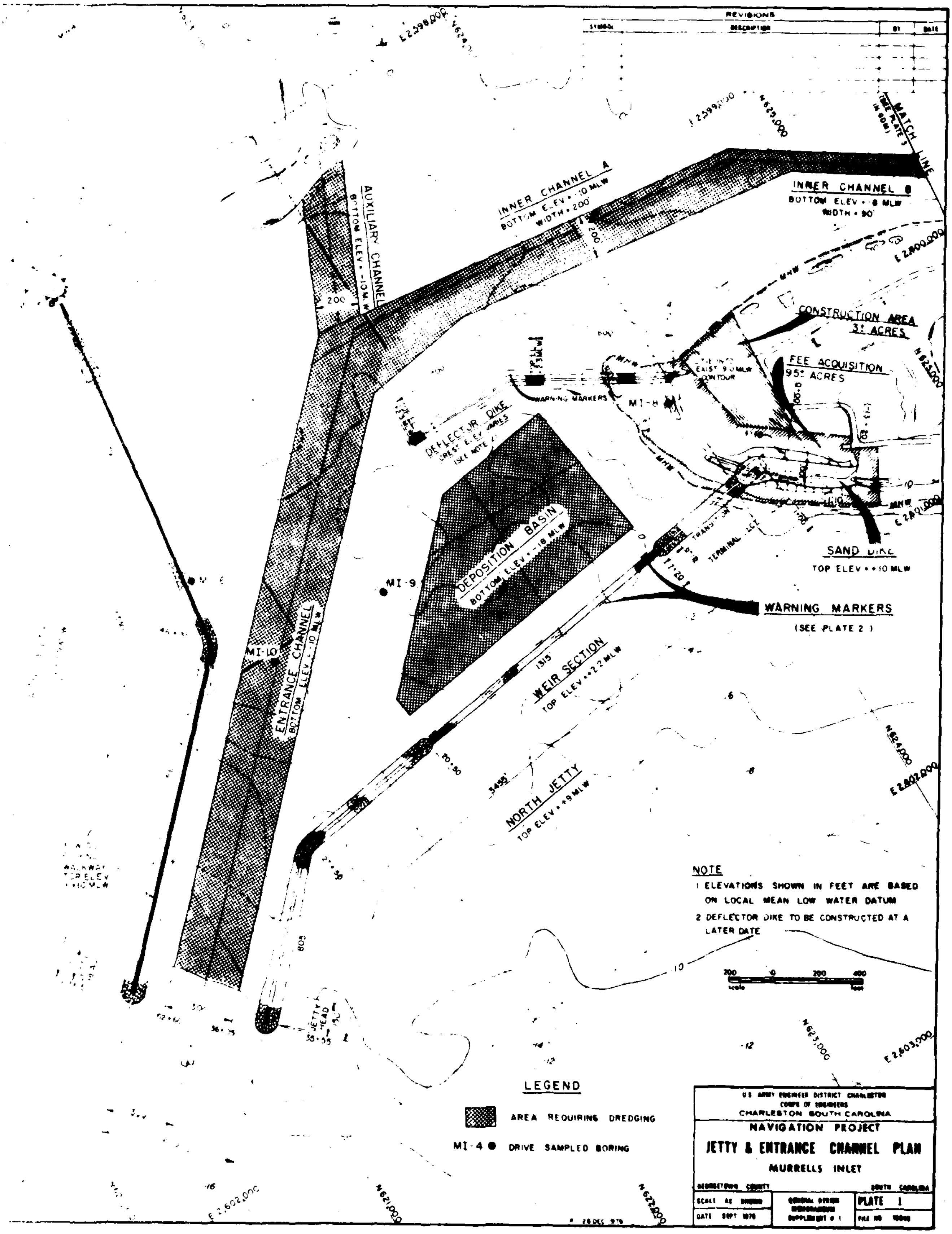
NORTH CAROLINA
SOUTH CAROLINA
GEORGIA
ATLANTIC OCEAN
VICINITY MAP
PIPELINE EASEMENT
DISPOSAL AREA
CAPTAIN ALEX'S MARINA
CAPTAIN DICK'S MARINA
FORMER ARMY CRASH BOAT DOCK
TURNING BASIN
PERRY'S LANDING
CREEK
INNER
MAIN
DEFLECTOR DIKE
SAND DIKE
NOURISHMENT AREA
WEIR
NORTH JETTY
PIPELINE EASEMENT (TEMPORARY)
GARDEN CITY BEACH
GULF STREAM MARINA
ATLANTIC
OCEAN
SCALE IN FEET
800 0 800 1600 2400
CHARLESTON, SOUTH CAROLINA
NAVIGATION PROJECT
GENERAL PLAN
MURRELLS INLET
PLATE 1A

2A/2


MI-1
MI 4
MI-3
MI
MI-5
MI-6
SAND DIKE
PERPETUAL EASEMENT
SOUTH JETTY
TOP ELEV +9 MLW
8' WIDE FISHING WALKWAY TOP ELEV +10 MLW
1580'
150' JETTY HEAD
62+15
300'
600'
E 2,600,000
E 2,601,000
N620,000

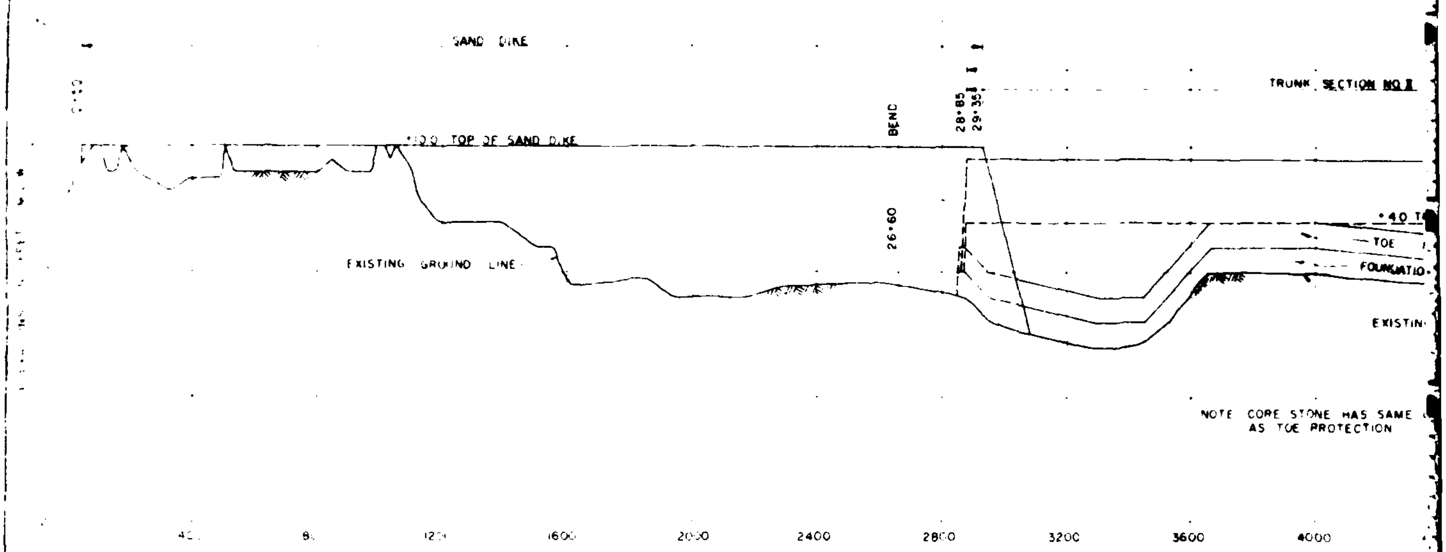
PLATE 1

1 of 2

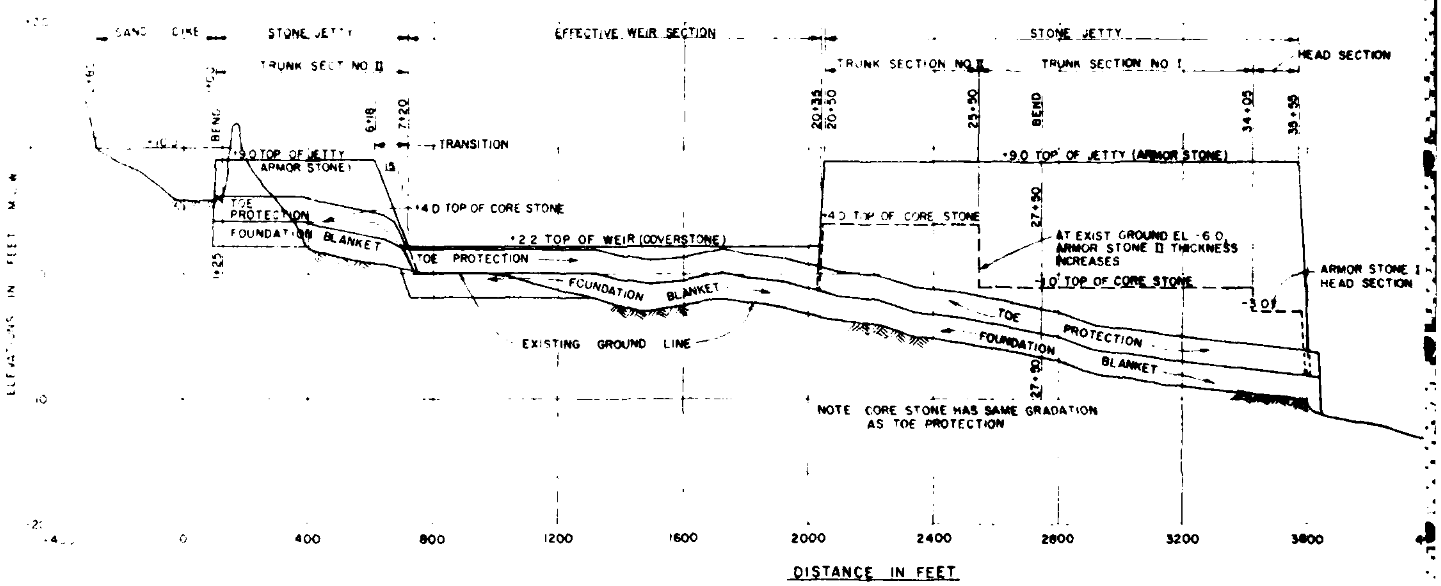


REVISIONS
INNER CHANNEL A
INNER CHANNEL B
AUXILIARY CHANNEL
MATCH LINE
CONSTRUCTION AREA
FEE ACQUISITION
95± ACRES
DEFLECTOR DIKE
WARNING MARKERS
DEPOSITION BASIN
BOTTOM ELEV -18 MLW
ENTRANCE CHANNEL
SAND DIKE
TOP ELEV +10 MLW
WARNING MARKERS
(SEE PLATE 2)
WEIR SECTION
NORTH JETTY
NOTE
1 ELEVATIONS SHOWN IN FEET ARE BASED ON LOCAL MEAN LOW WATER DATUM
2 DEFLECTOR DIKE TO BE CONSTRUCTED AT A LATER DATE
LEGEND
AREA REQUIRING DREDGING
MI-4 ● DRIVE SAMPLED BORING
CHARLESTON SOUTH CAROLINA
NAVIGATION PROJECT
JETTY & ENTRANCE CHANNEL PLAN
MURRELLS INLET
PLATE 1

2 of 2



PROFILE

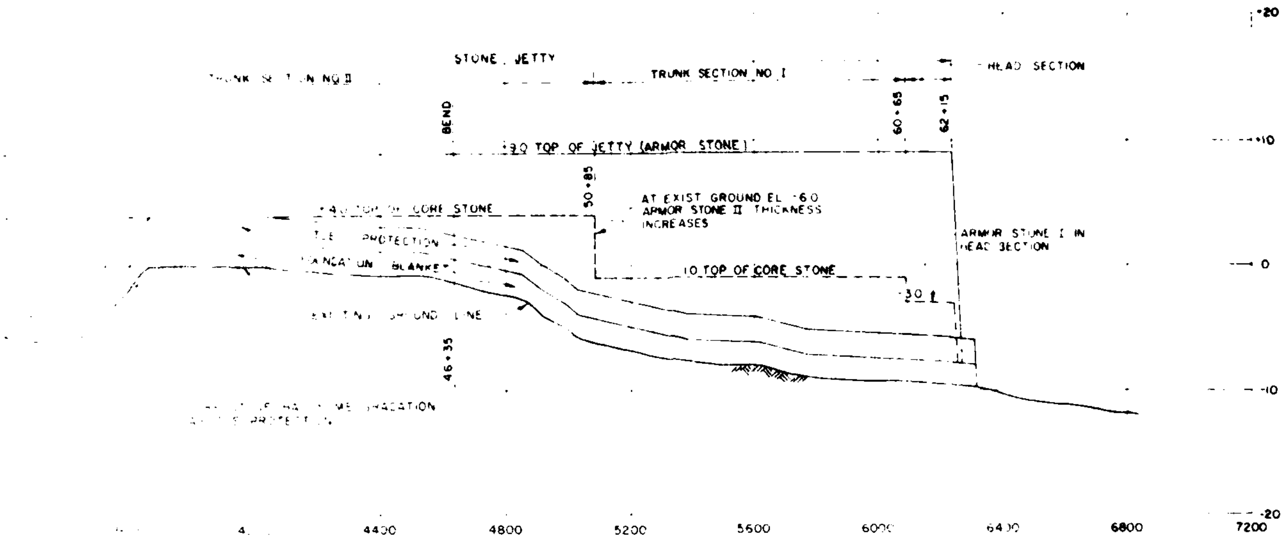


NORTH JETTY PROFILE

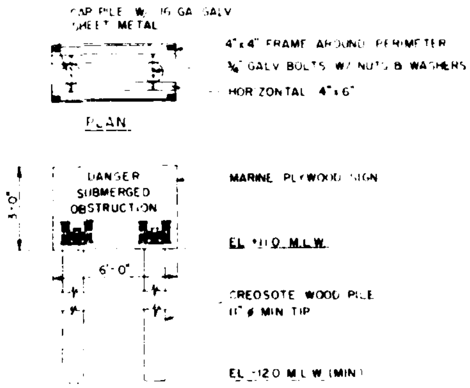
PLATE 2

1 of 2

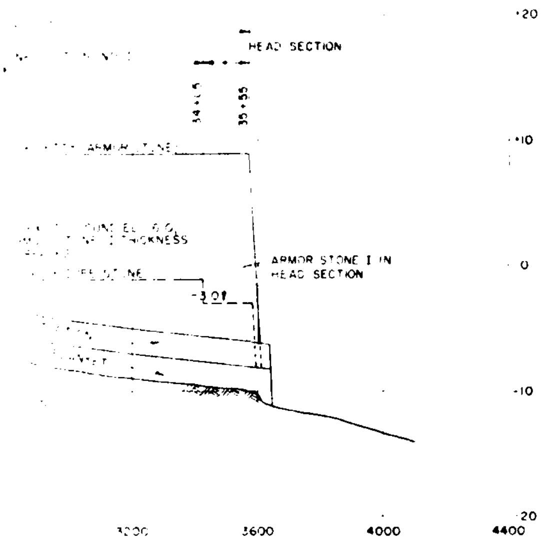
REVISIONS			
SYMBOL	DESCRIPTION	BY	DATE



JETTY PROFILE



WARNING MARKER
AT WEIR SECTION
N.T.S.



U.S. ARMY ENGINEER DISTRICT CHARLESTON
CORPS OF ENGINEERS
CHARLESTON SOUTH CAROLINA

NAVIGATION PROJECT

JETTY PROFILES

MURRELLS INLET

GEORGETOWN COUNTY — SOUTH CAROLINA

SCALE AS SHOWN	GENERAL DESIGN MEMORANDUM	PLATE 2
DATE SEPT 1976	SUPPLEMENT # 1	FILE NO

R: 28 DEC 976

2 of 2

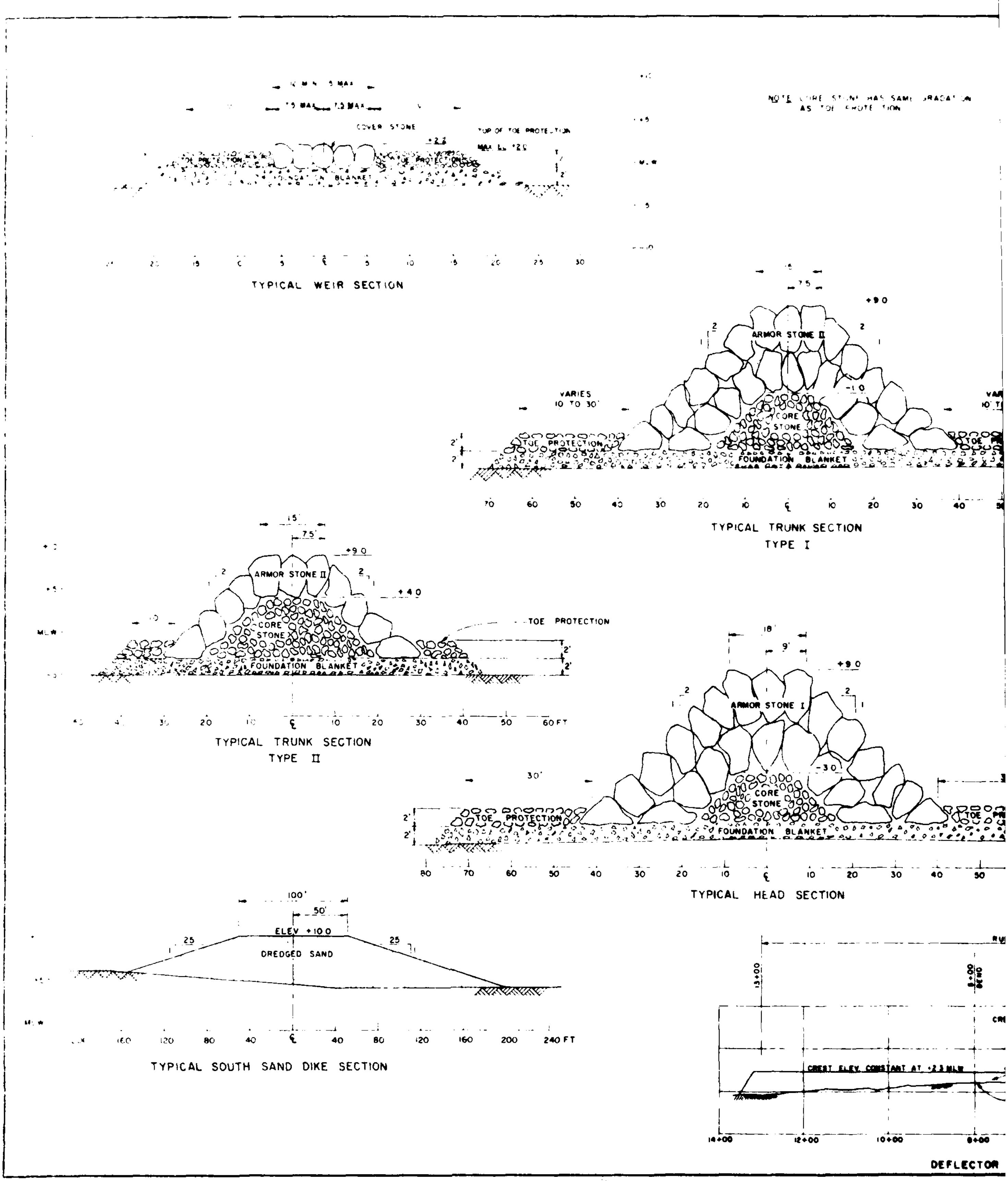

COVER STONE
TOE PROTECTION
FOUNDATION BLANKET
TYPICAL WEIR SECTION
ARMOR STONE II
VARIES
10 TO 30'
CORE STONE
+9.0
-1.0
TYPICAL TRUNK SECTION
TYPE I
+4.0
TYPICAL TRUNK SECTION
TYPE II
ARMOR STONE I
-3.0
30'
TYPICAL HEAD SECTION
ELEV +10.0
DREDGED SAND
TYPICAL SOUTH SAND DIKE SECTION
CREST ELEV. CONSTANT AT +2.3 MLW
14+00
12+00
10+00
8+00
DEFLECTOR

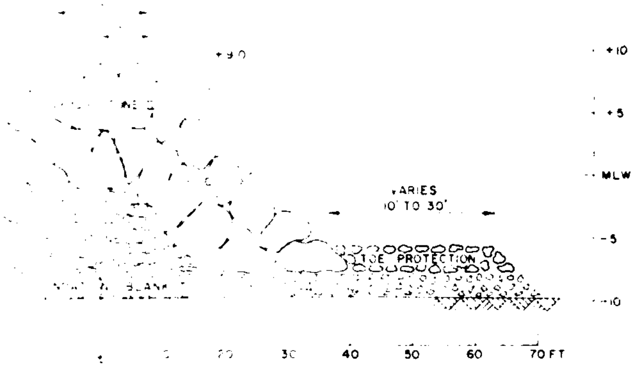
PLATE 3

1 of 2

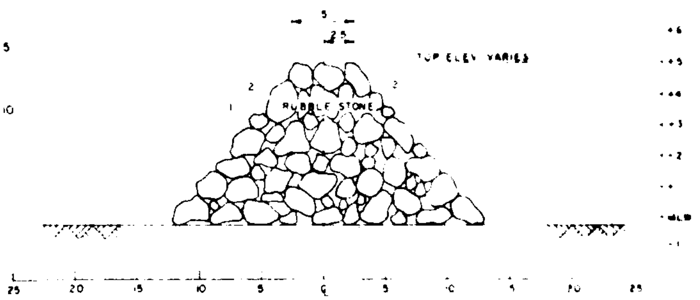
REVISIONS

SYMBOL	DESCRIPTION	BY	DATE

TYPICAL SECTION ... NOURISHMENT AREAS



TRUNK SECTION



TYPICAL DEFLECTOR DIKE SECTION

JETTY STONE *

TYPE	WEIGHT (TONS)	SIZE (INCHES)
ARMOR I	6 — 10	60 — 70
ARMOR II	4 — 7	50 — 60
CORE	———	12.0 — 24.0
TOE PROTECTION	———	12.0 — 24.0
FOUNDATION BLANKET	———	0.25 — 6.0

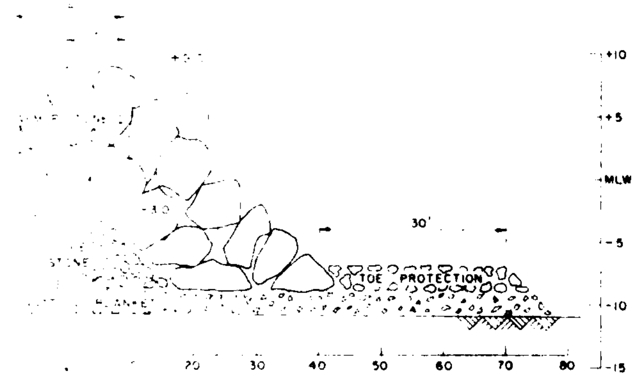
* ASSUMED SPECIFIC WEIGHT 160 LBS PER CUBIC FOOT (DRY)

WEIR STONE *

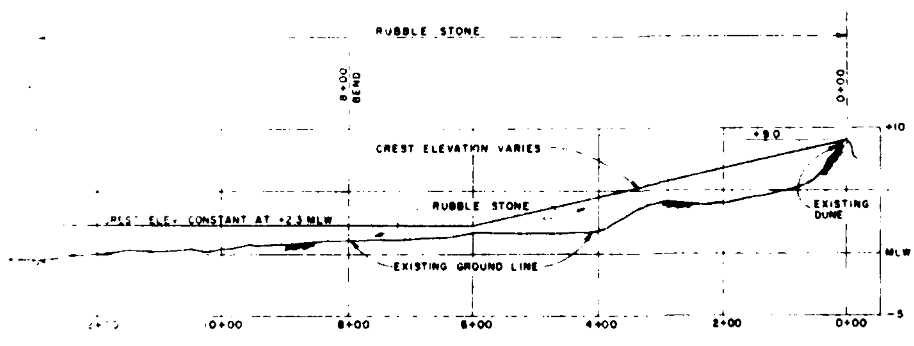
TYPE	WEIGHT (LBS)	SIZE (INCHES)
COVER STONE	2400 — 5000	30 — 42
TOE PROTECTION	100 — 900	12 — 24
CORE STONE	100 — 900	12 — 24
FOUNDATION BLANKET	———	0.25 — 6.0

DEFLECTOR DIKE STONE *

TYPE	WEIGHT (LBS)	SIZE (INCHES)
RUBBLE STONE	100 — 900	12 - 24



SECTION



DEFLECTOR DIKE PROFILE

R: 26 DEC 1976

U.S. ARMY ENGINEER DISTRICT CHARLESTON
CORPS OF ENGINEERS
CHARLESTON SOUTH CAROLINA

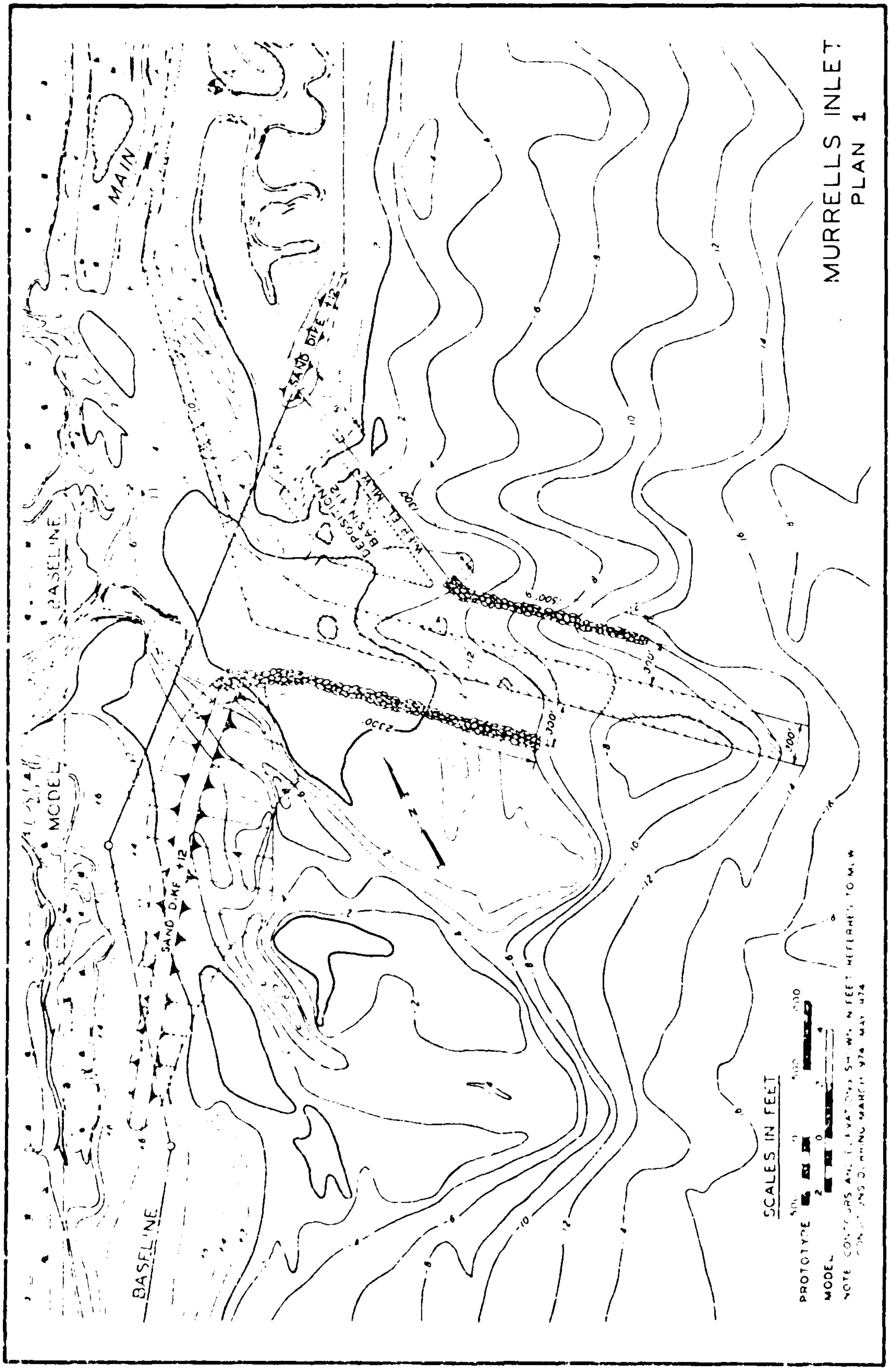
NAVIGATION PROJECT
JETTY & WEIR SECTIONS
AND DEFLECTOR DIKE DETAILS
MURRELLS INLET

GEORGETOWN COUNTY — SOUTH CAROLINA

SCALE AS SHOWN	GENERAL DESIGN MEMORANDUM SUPPLEMENT NO. 1	PLATE 3
DATE SEPT 1976		FILE NO. [illegible]

2 of 2

FIGURES



MURRELLS INLET
PLAN 1
MAIN
BASELINE
MODEL
SAND DIKE +12
BASELINE
SCALES IN FEET
PROTOTYPE
MODEL

FIGURE 1

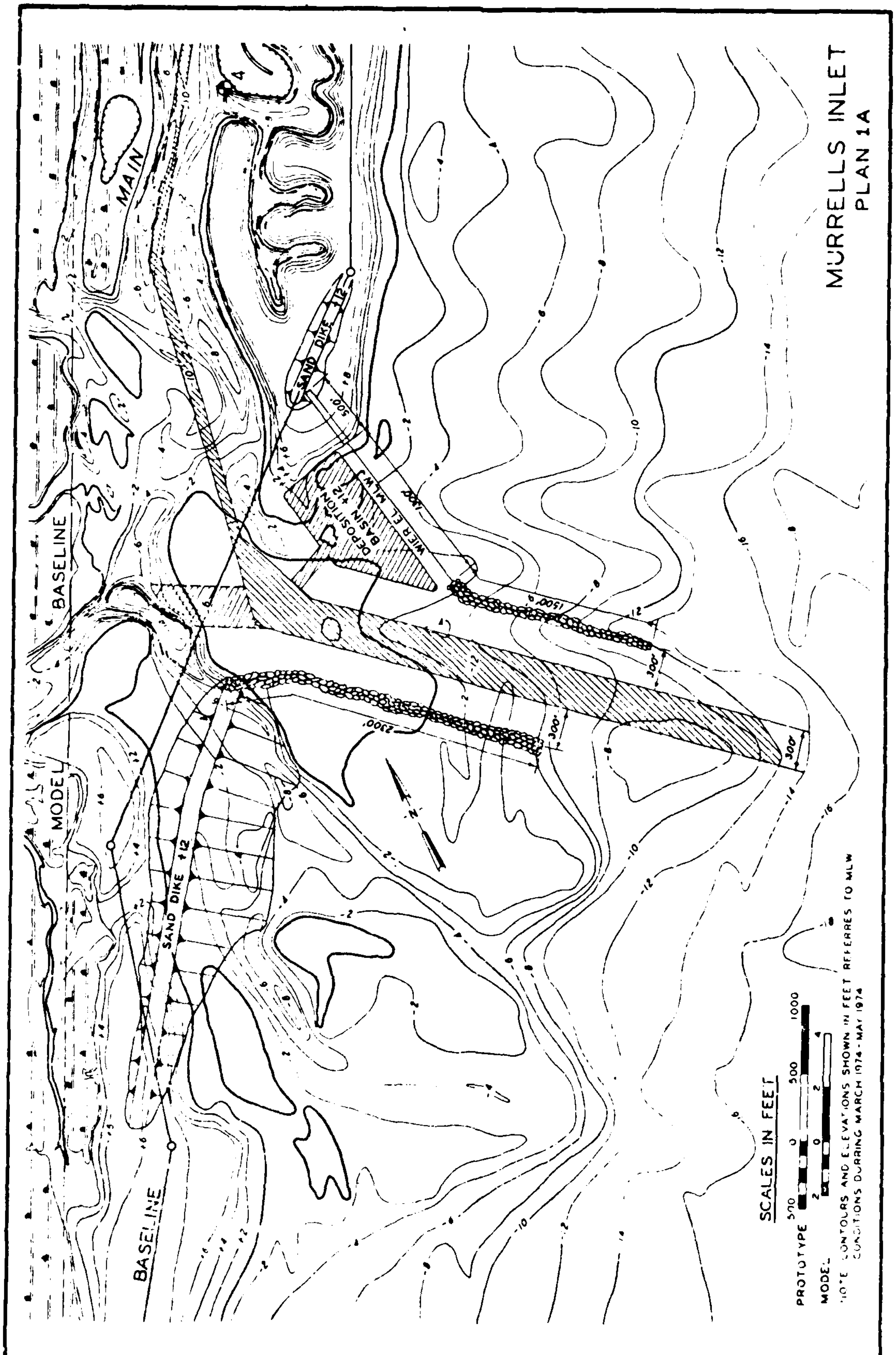

MURRELLS INLET
PLAN 1A
BASELINE
MAIN
MODEL
BASELINE
SAND DIKE +12
SAND DIKE +12
WIER EL MLW
DEPOSITION BASIN +12
SCALES IN FEET
PROTOTYPE 500 0 500 1000
MODEL 2 0 2 4
NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MLW
CONDITIONS DURING MARCH 1974 - MAY 1974

FIGURE 2

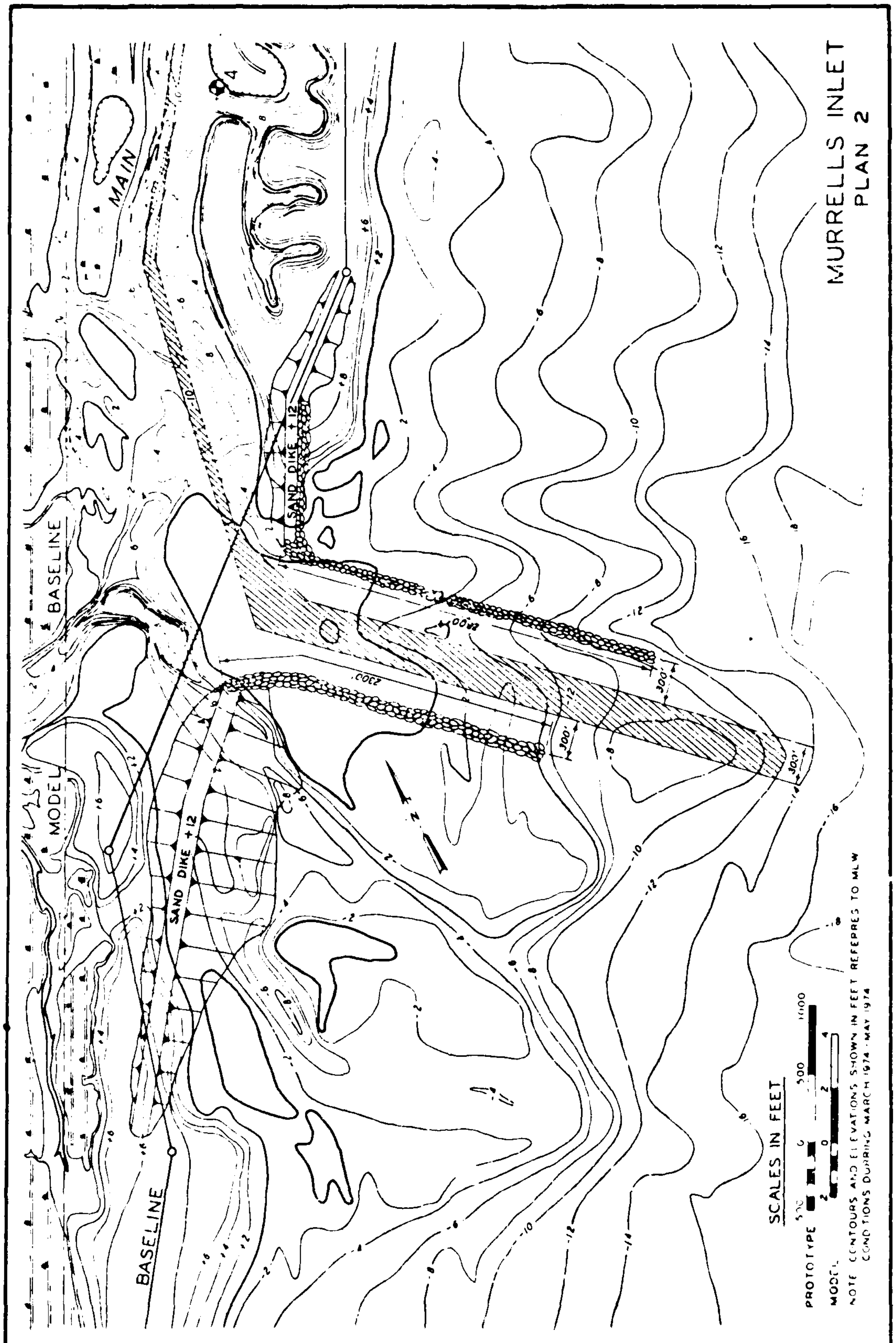

MURRELLS INLET
PLAN 2
SCALES IN FEET
PROTOTYPE
MODEL
SAND DIKE +12
SAND DIKE +12
BASELINE
BASELINE
MAIN
MODEL

FIGURE 3

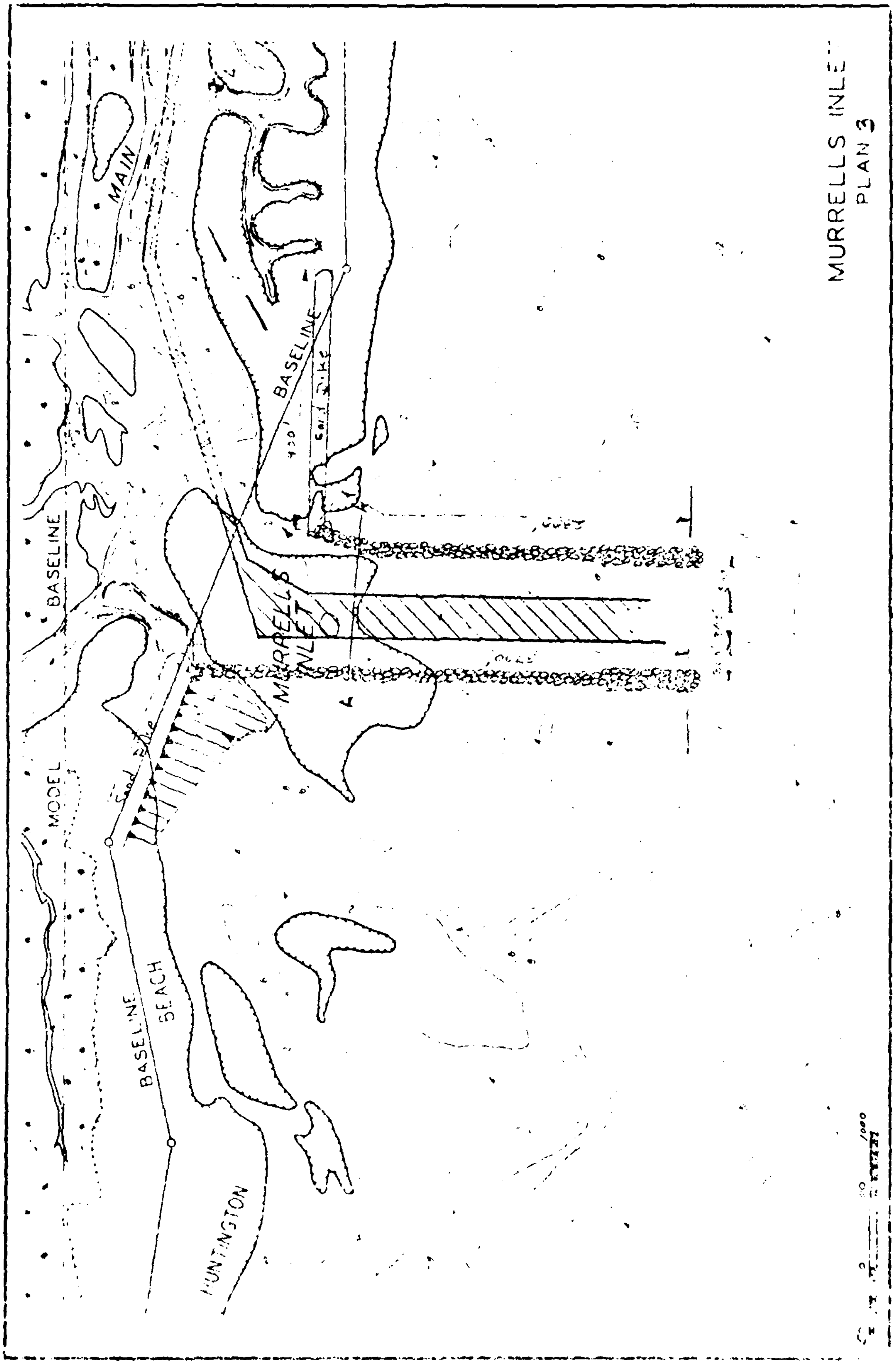

MURRELLS INLET
PLAN 3
BASELINE
MAIN
BASELINE
MURRELLS INLET
MODEL
Sand Dike
BASELINE
BEACH
HUNTINGTON

FIGURE 4

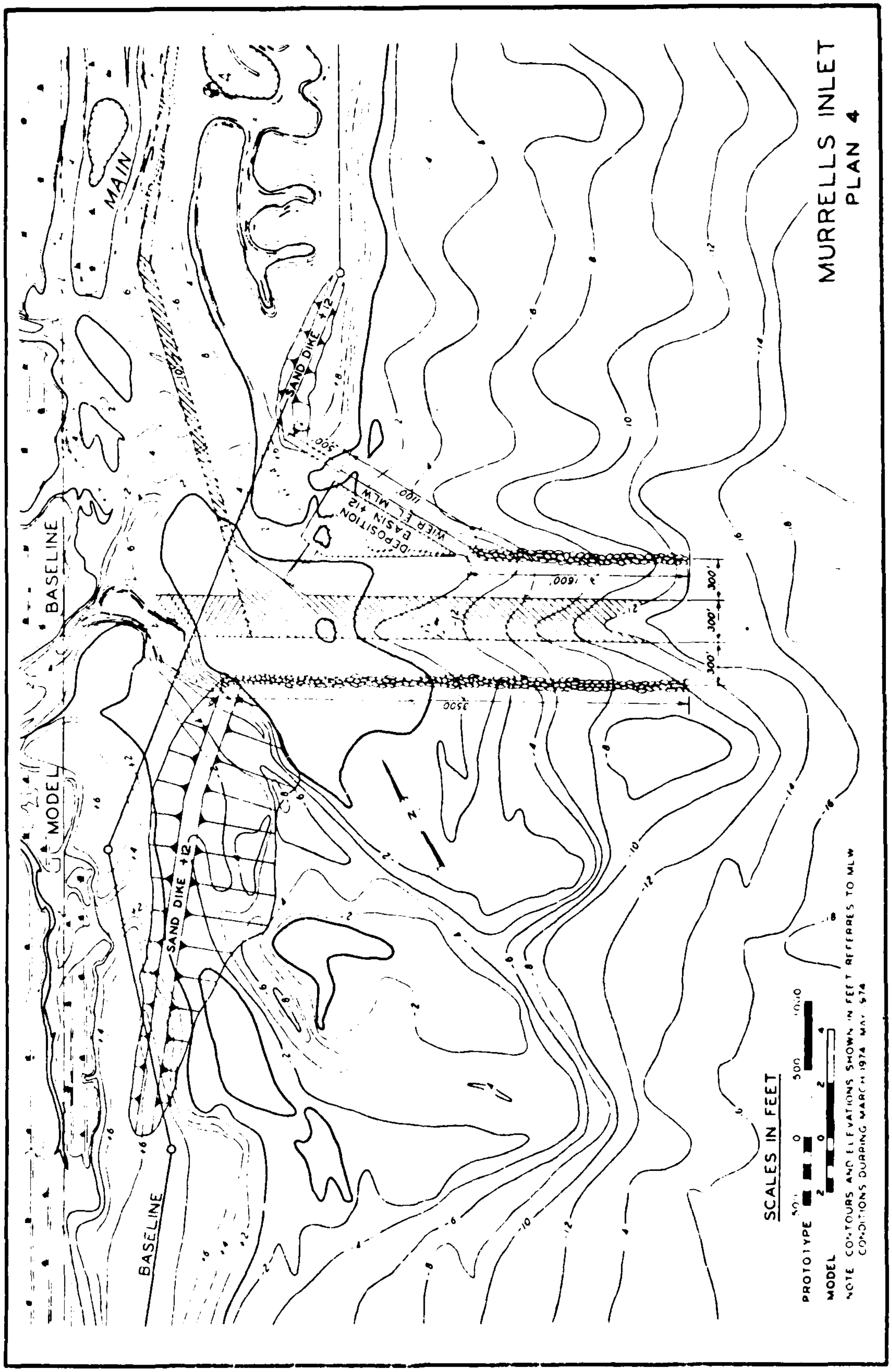

MURRELLS INLET
PLAN 4
SCALES IN FEET
PROTOTYPE
MODEL
NOTE CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MLW
CONDITIONS DURING MARCH 1974 MAY 1974
MODEL BASELINE
BASELINE
MAIN
SAND DIKE +12
SAND DIKE +12
DEPOSITION BASIN +12
WIER EL MLW
300'
300'
300'
1600'
1100'
3500'

FIGURE 5

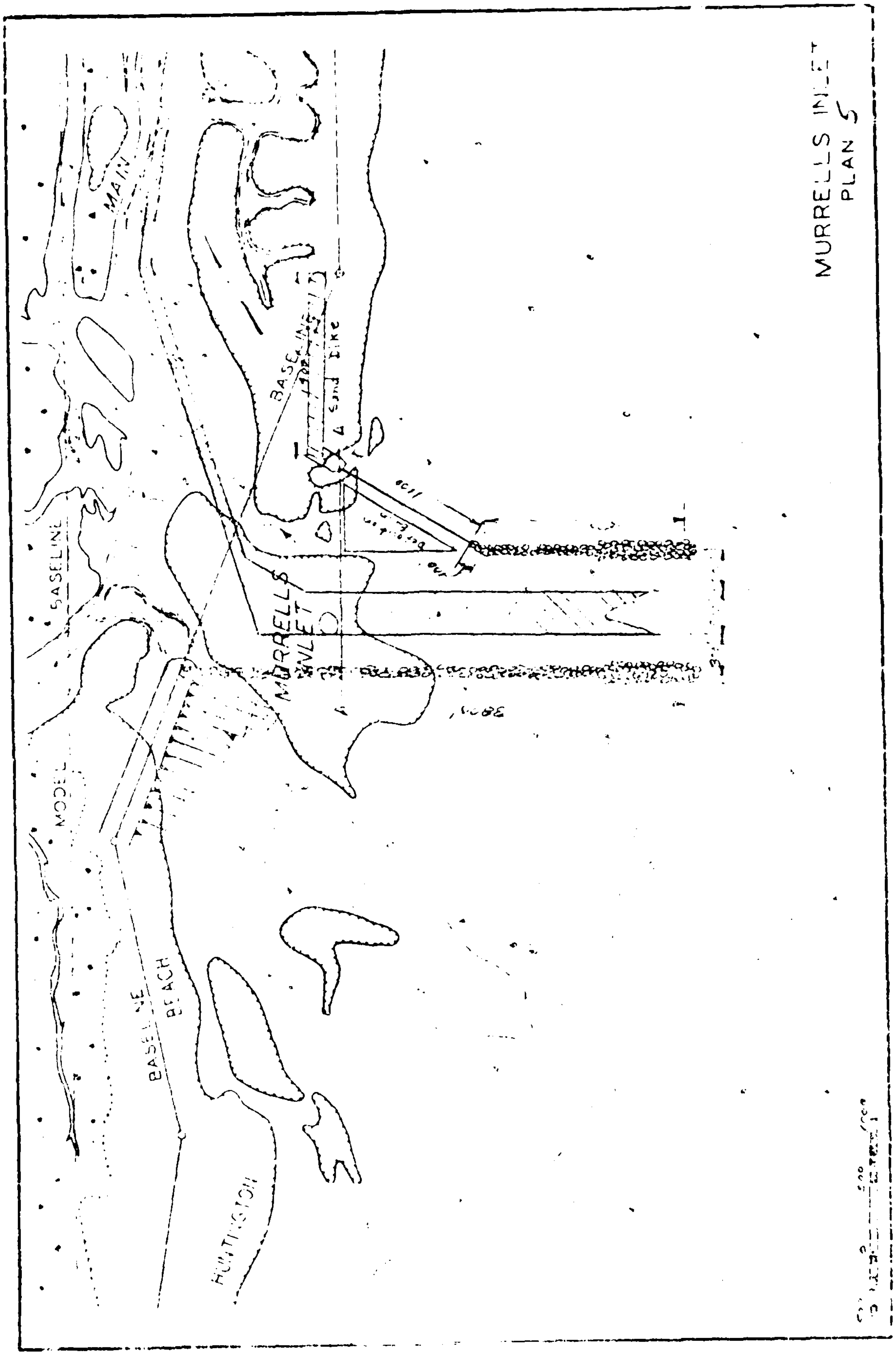

MURRELLS INLET
PLAN 5
MAIN
BASELINE
BASELINE
sand dike
MURRELLS INLET
MODEL
BASELINE
BEACH
HUNTINGTON

FIGURE 6

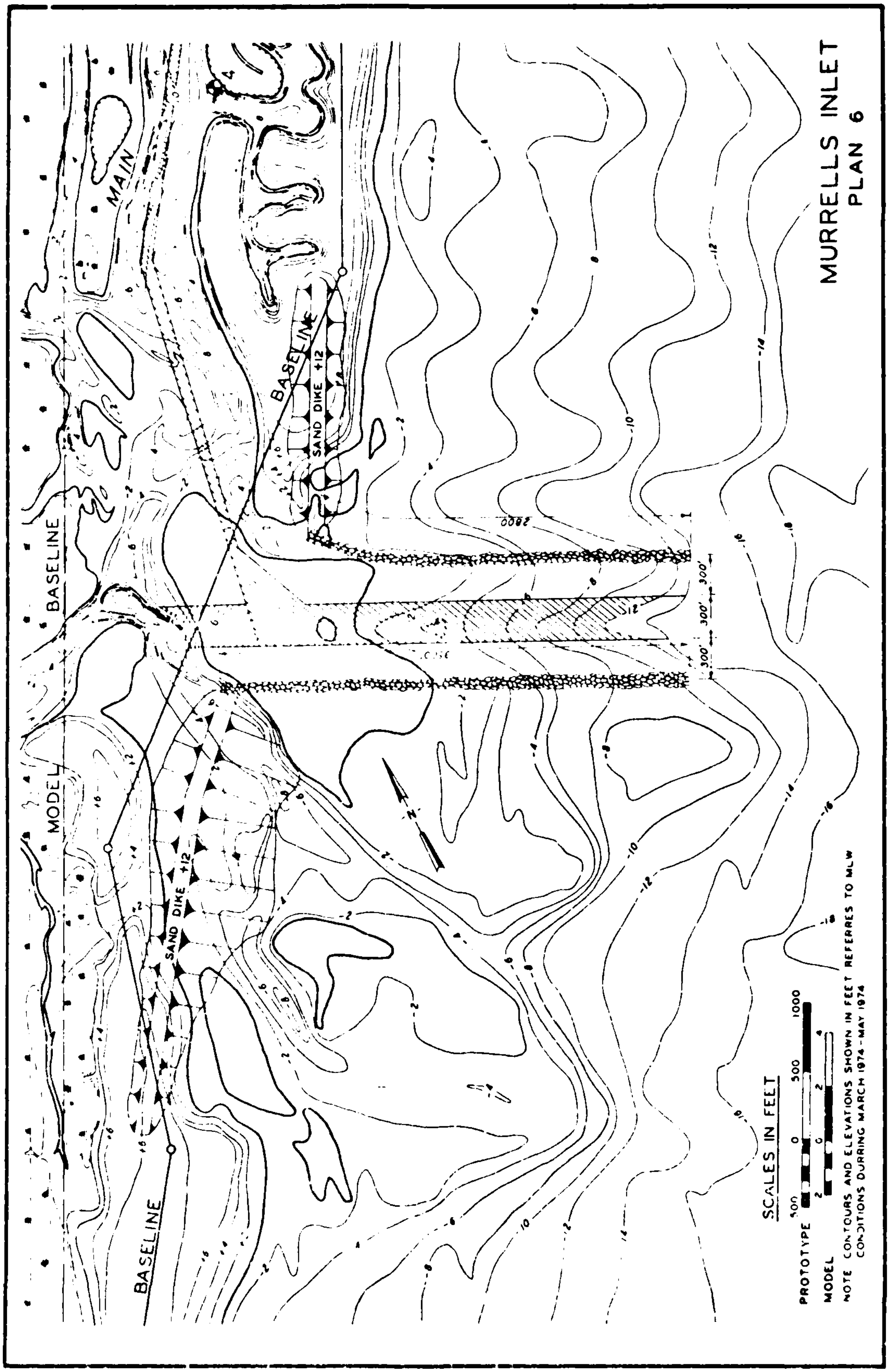

MURRELLS INLET
PLAN 6
BASELINE
BASELINE
BASELINE
MODEL
MAIN
SAND DIKE +12
SAND DIKE +12
SCALES IN FEET
PROTOTYPE 500 0 500 1000
MODEL 2 0 2 4
NOTE CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRES TO MLW
CONDITIONS DURRING MARCH 1974-MAY 1974

FIGURE 7

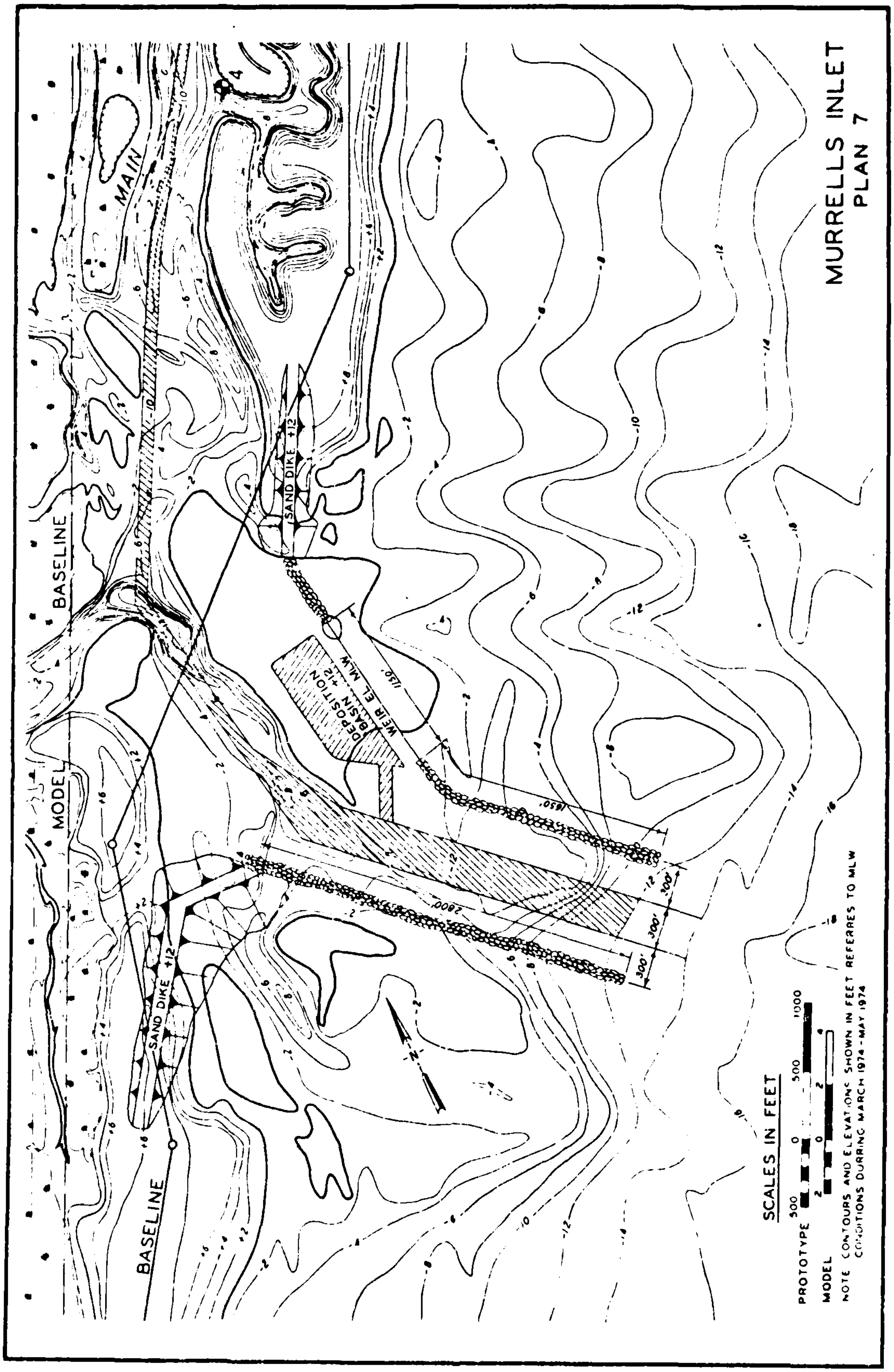
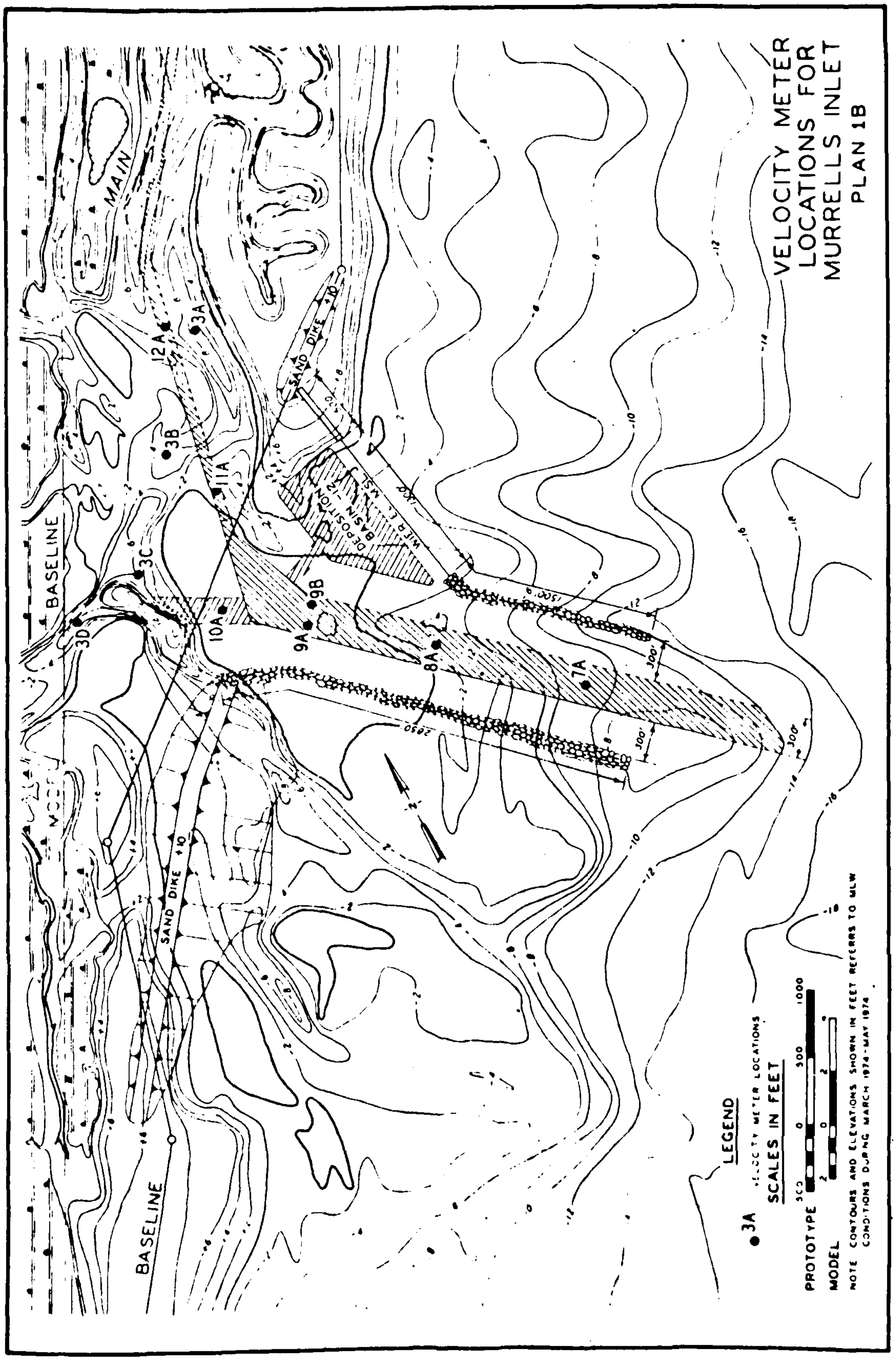

MURRELLS INLET
PLAN 7
BASELINE
MODEL
BASELINE
MAIN
SAND DIKE +12
SAND DIKE +12
WEIR EL MLW
DEPOSITION BASIN +12
SCALES IN FEET
PROTOTYPE
MODEL

FIGURE 8



VELOCITY METER LOCATIONS FOR MURRELLS INLET
PLAN 1B
BASELINE
MAIN
SAND DIKE +10
DEPOSITION BASIN
3A
3B
3C
3D
7A
8A
9A
9B
10A
11A
12A
LEGEND
SCALES IN FEET
PROTOTYPE
MODEL

FIGURE 9

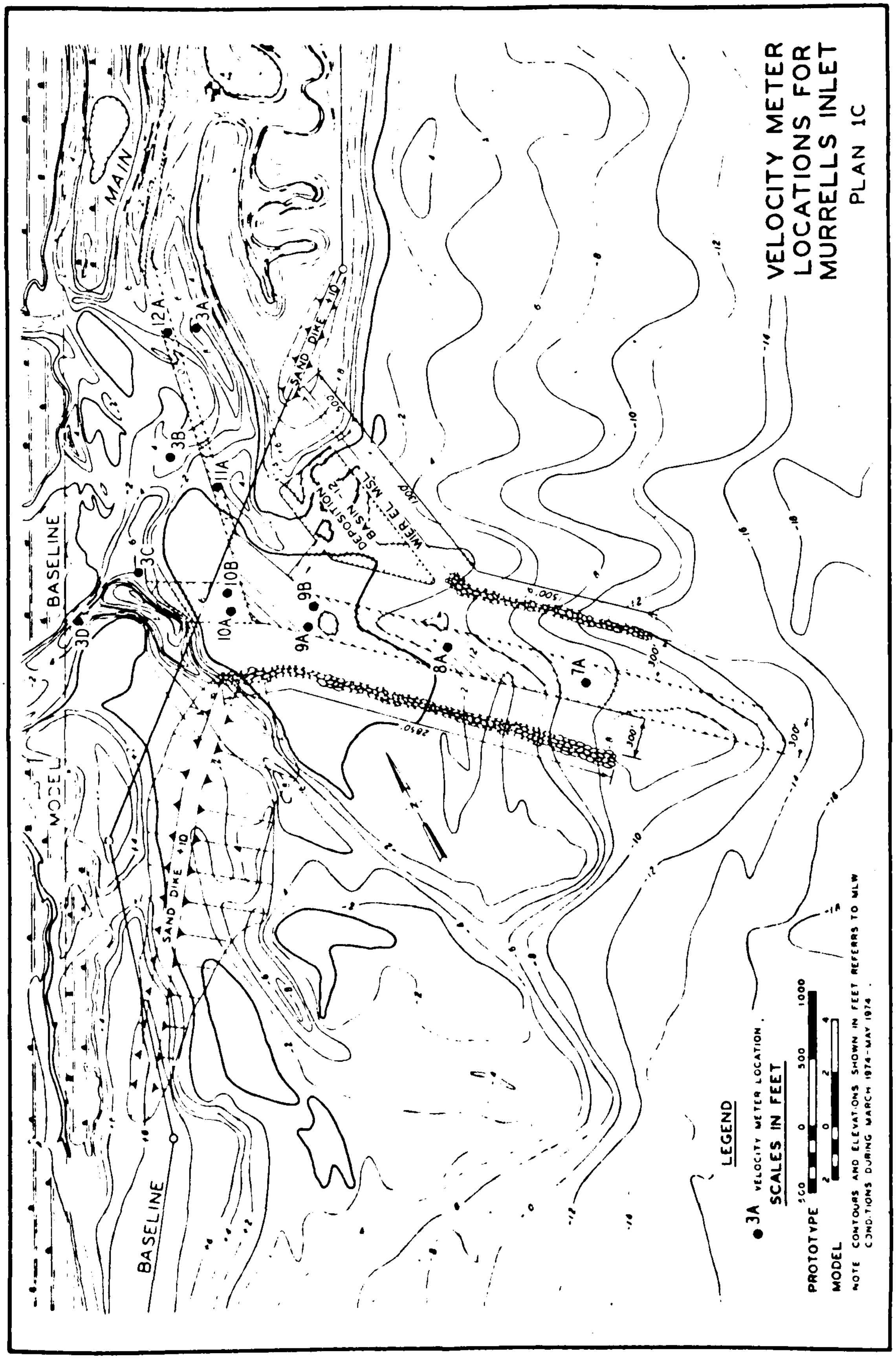

VELOCITY METER LOCATIONS FOR MURRELLS INLET
PLAN 1C
LEGEND
3A VELOCITY METER LOCATION
SCALES IN FEET
PROTOTYPE 500 0 500 1000
MODEL 2 0 2 4
NOTE CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRS TO MLW CONDITIONS DURING MARCH 1974-MAY 1974.
BASELINE
MODEL
MAIN
SAND DIKE +10
WIER EL MSL
DEPOSITION BASIN
3A
3B
3C
3D
7A
8A
9A
9B
10A
10B
11A
12A

FIGURE 10

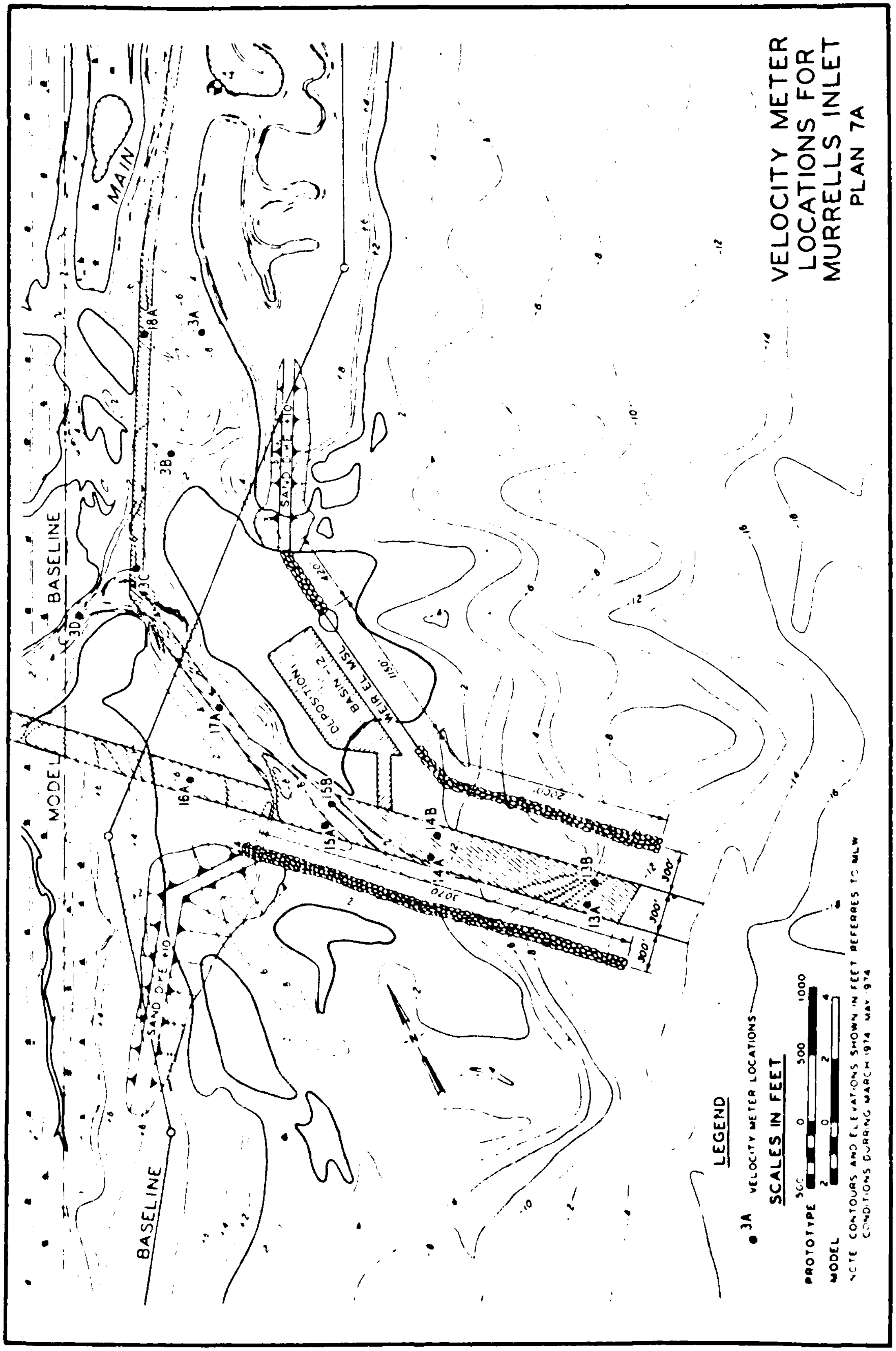

VELOCITY METER
LOCATIONS FOR
MURRELLS INLET
PLAN 7A
LEGEND
3A VELOCITY METER LOCATIONS
SCALES IN FEET
PROTOTYPE 500 0 500 1000
MODEL 2 0 2 4
NOTE CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MLW CONDITIONS DURING MARCH 1974 MAY 1974
BASELINE
BASELINE
MODEL
MAIN
WEIR EL MSL
DEPOSITION BASIN -12
SAND DIKE +10
SAND DIKE +10
3A
3B
3C
3D
13A
13B
14A
14B
15A
15B
16A
17A
18A
3070'
2000'
1150'
420'
300'
300'
300'
300'

FIGURE 11

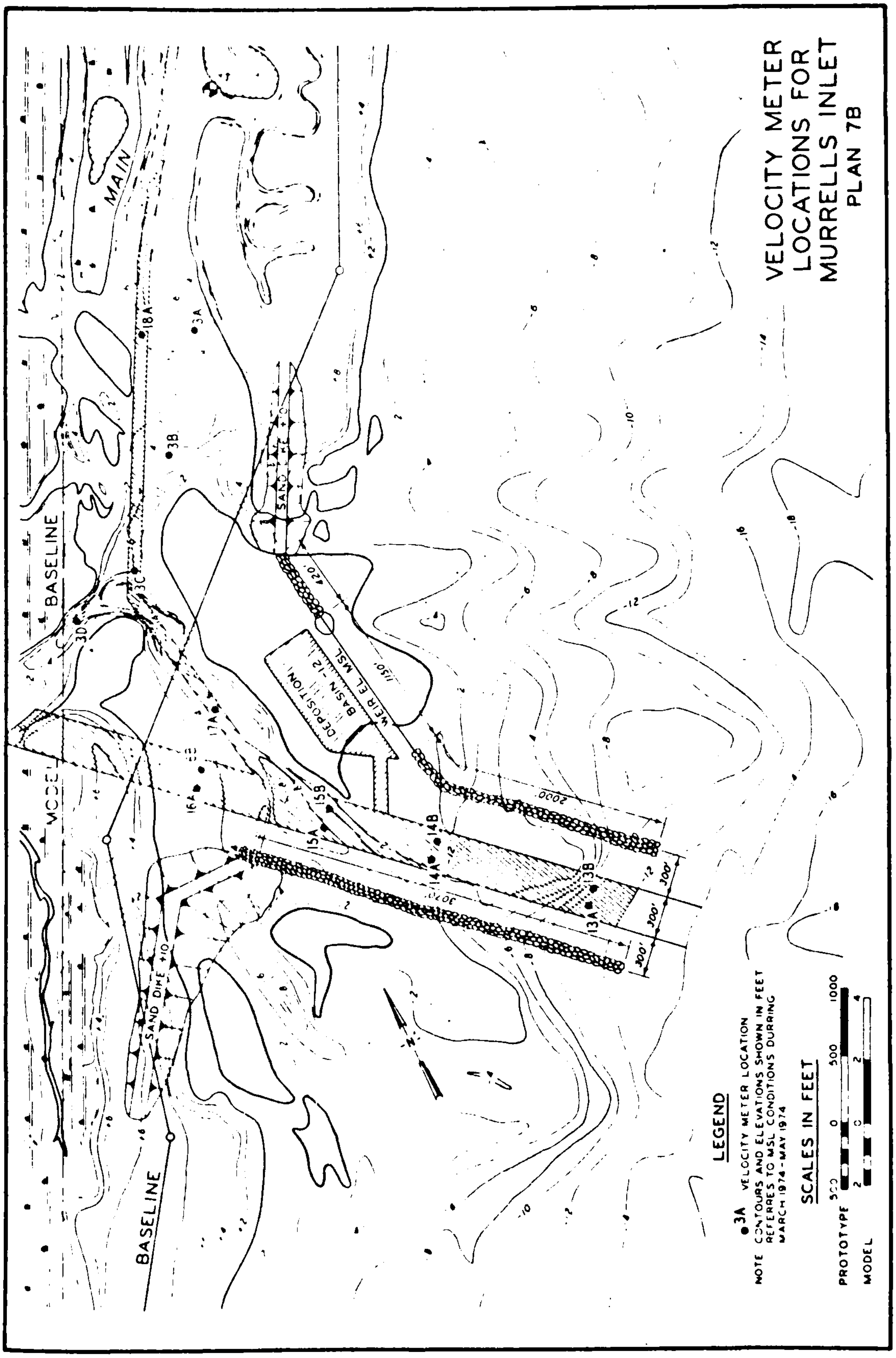

VELOCITY METER
LOCATIONS FOR
MURRELLS INLET
PLAN 7B
LEGEND
3A VELOCITY METER LOCATION
NOTE CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MSL CONDITIONS DURING MARCH 1974 - MAY 1974
SCALES IN FEET
PROTOTYPE 500 0 500 1000
MODEL 2 0 2 4
BASELINE
BASELINE
MAIN
MODEL
SAND DIKE +10
DEPOSITION BASIN -12
WEIR EL MSL
3A
3B
3C
3D
13A
13B
14A
14B
15A
15B
16A
17A
18A
300'
300'
300'
2000'

FIGURE 12

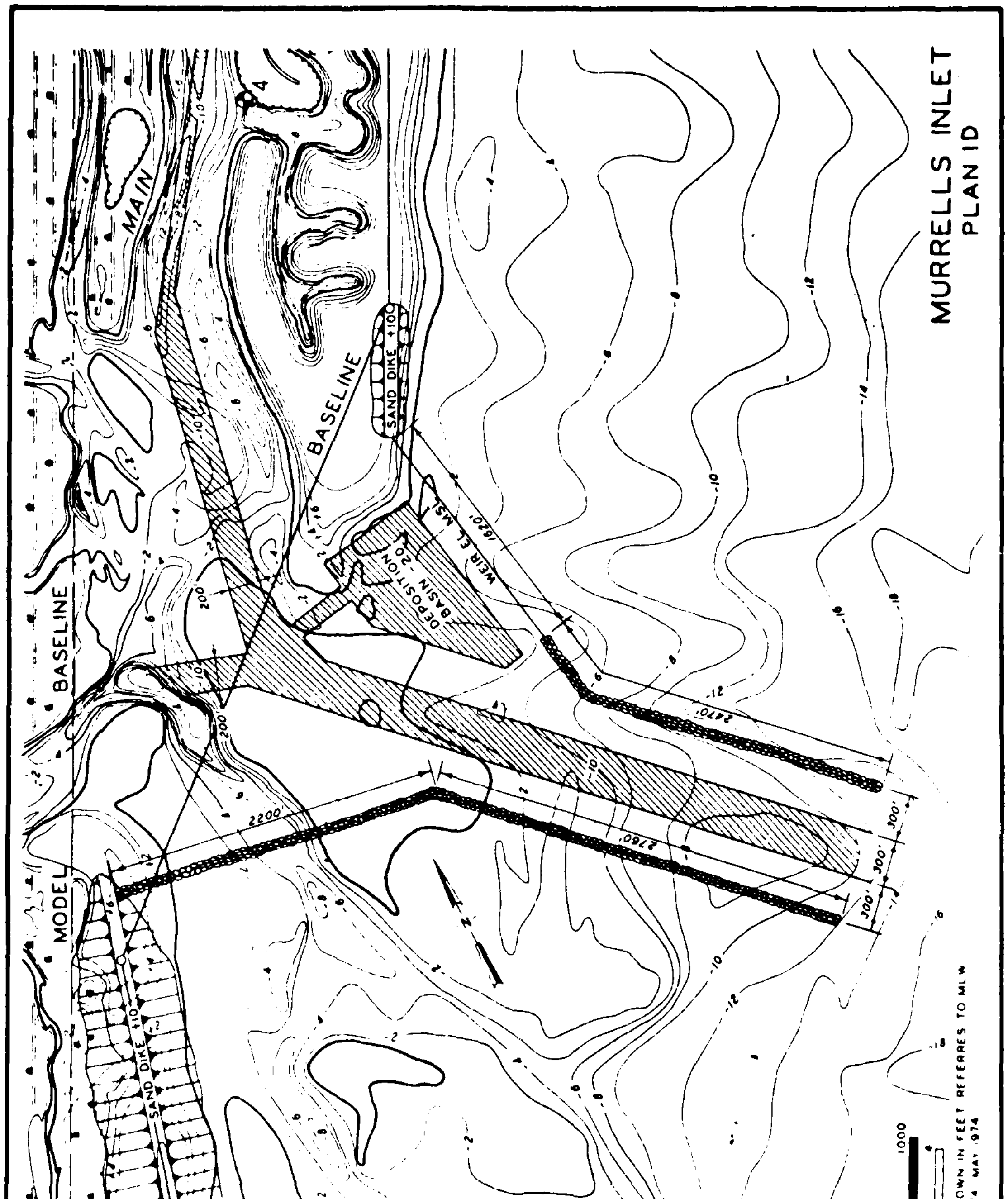

MURRELLS INLET
PLAN 1D
SCALE IN FEET
PROTOTYPE 500 0 500 1000
MODEL 2 0 2 4
BASELINE
MODEL
MAIN
SAND DIKE +10
DEPOSITION BASIN 20
WEIR EL MSL
1020'
2200'
2760'
2470'
300'

FIGURE 13

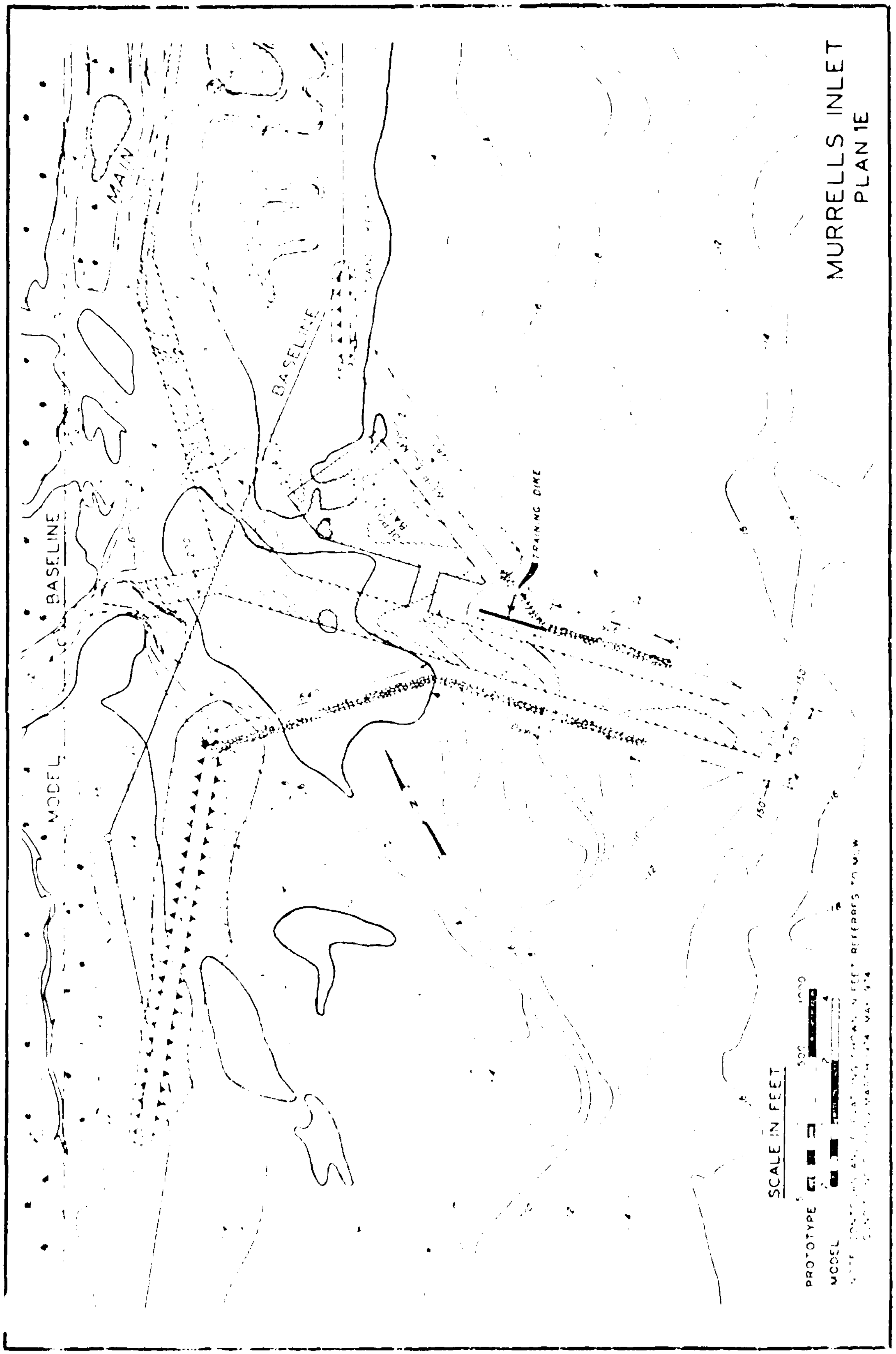

MURRELLS INLET
PLAN 1E
MAIN
BASELINE
MODEL BASELINE
TRAINING DIKE
SCALE IN FEET
PROTOTYPE
MODEL

FIGURE 14

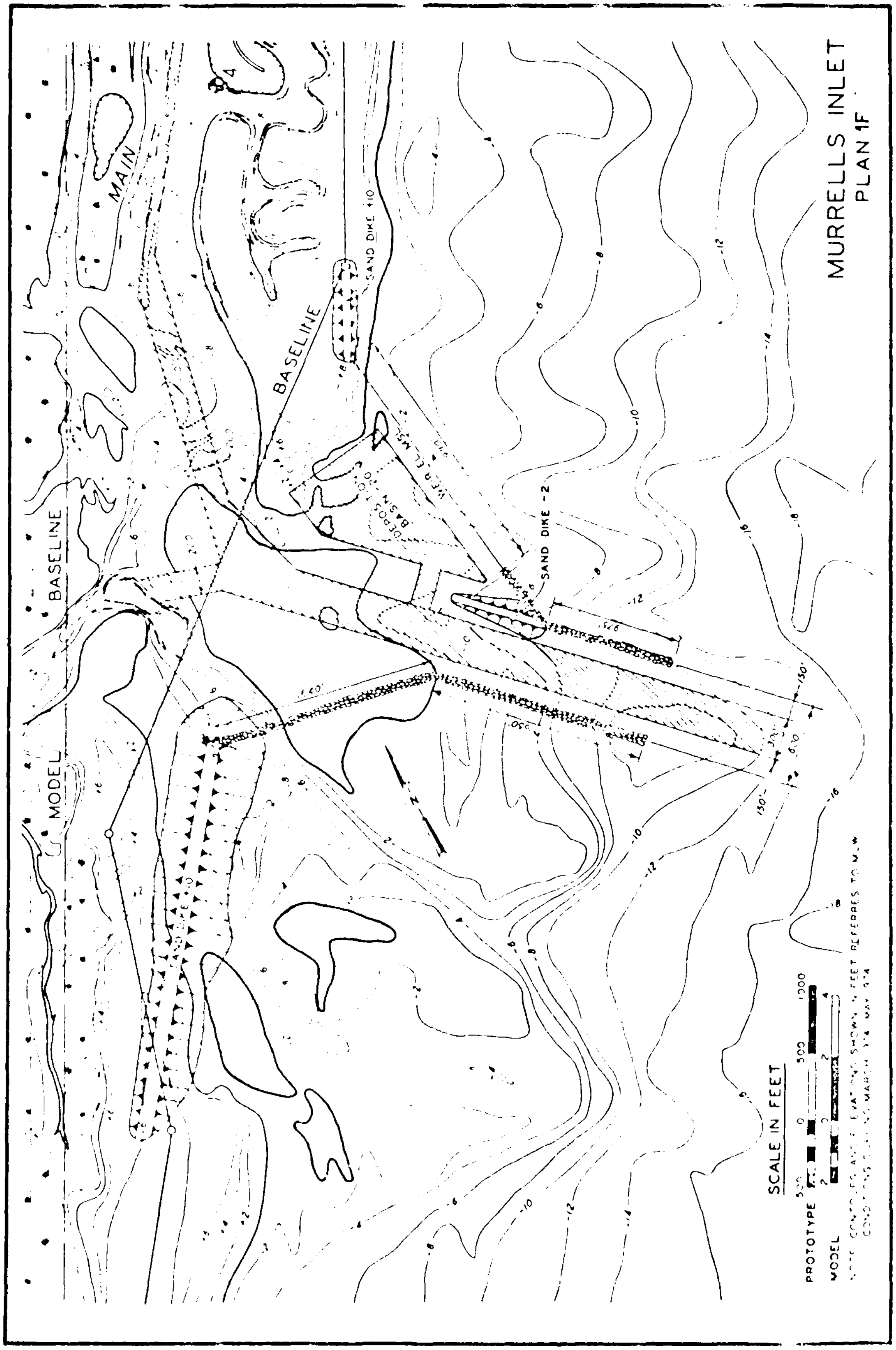

MURRELLS INLET
PLAN 1F
SCALE IN FEET
PROTOTYPE
MODEL
BASELINE
MODEL BASELINE
MAIN
SAND DIKE +10
SAND DIKE -2

FIGURE 15

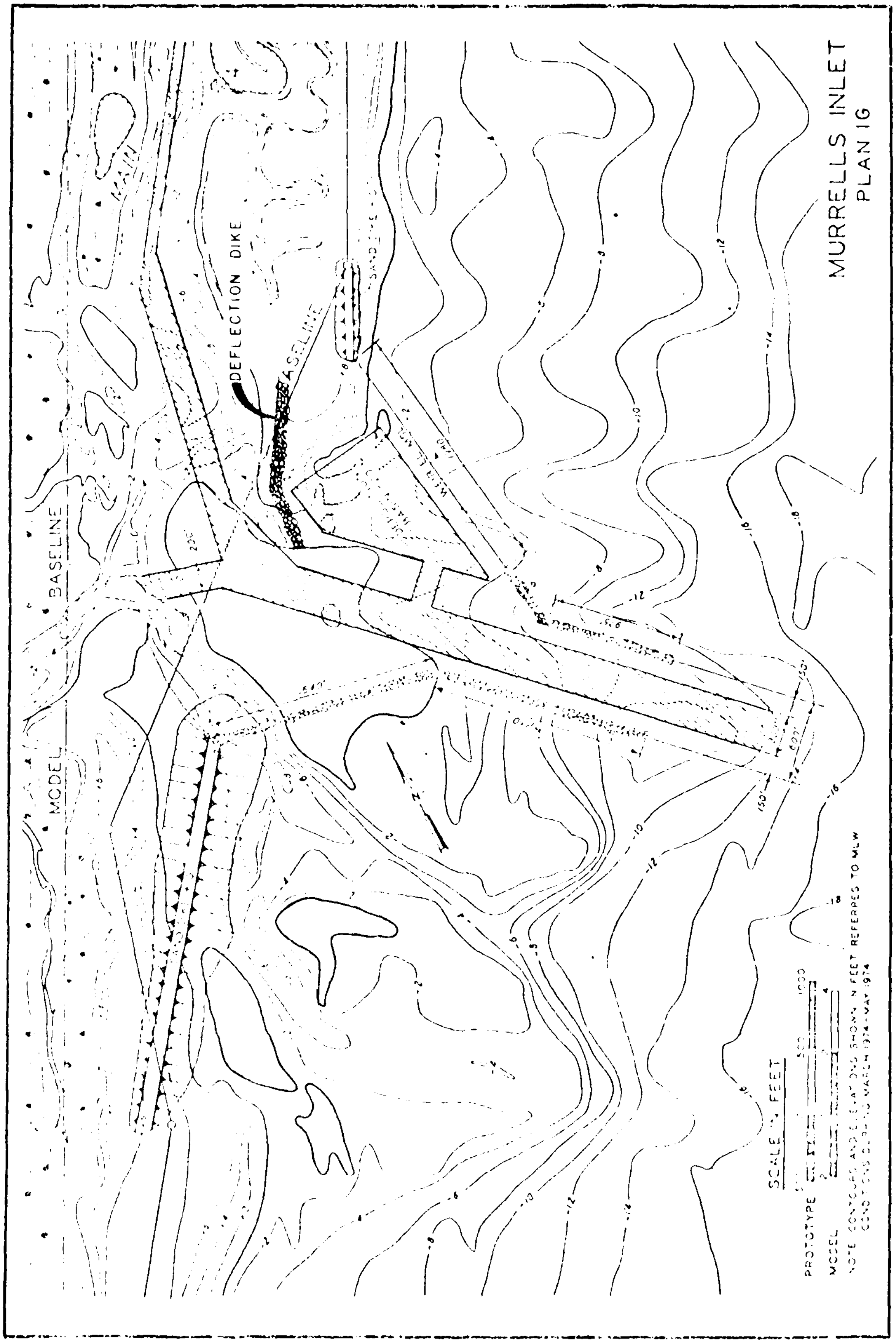

MURRELLS INLET
PLAN 1G
DEFLECTION DIKE
BASELINE
MAIN
MODEL BASELINE
SCALE IN FEET
PROTOTYPE
MODEL

FIGURE 16

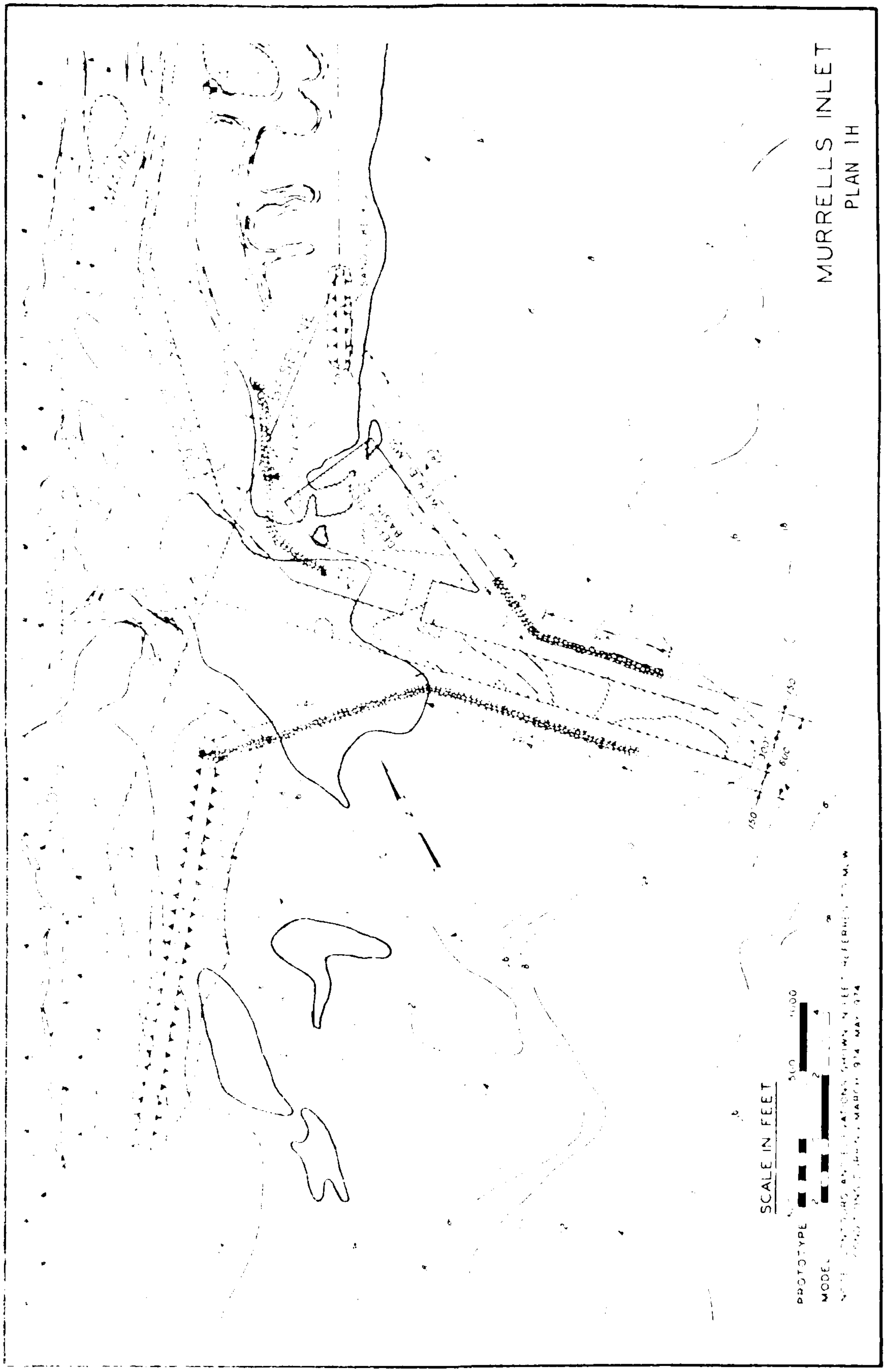


FIGURE 16A

VELOCITY SCALE

2 2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA

HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1B HOUR 7

VELOCITY SCALE

2 0 2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE
MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA
HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1B HOUR 10

VELOCITY SCALE

2 0 2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA

HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1C HOUR 7

FIGURE 19

VELOCITY SCALE

2 0 2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA

HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1C HOUR 10

FIGURE 20

VELOCITY SCALE
2 0 2 4 6 8

TEST CONDITIONS
ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE
MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA
HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 7A HOUR 7

FIGURE 21

VELOCITY SCALE

[illegible]

TEST CONDITIONS

[illegible] AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA
HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 7A HOUR 10

FIGURE 22

VELOCITY SCALE

SURFACE CURRENT
PATTERNS

PLAN 7B HOUR 7

FIGURE 25

VELOCITY SCALE

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE
MODEL OCEAN TIDE RANGE 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA
HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 7B HOUR 10

FIGURE 24

VELOCITY SCALE

TEST CONDITIONS
ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE
MODEL OCEAN TIDE RANGE 4.8 FEET

MURRELLS INLET SOUTH CAROLINA
HYDRAULIC MODEL STUDY
SURFACE CURRENT
PATTERNS
PLAN 1D HOUR 7

FIGURE 25

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE 4.8 FEET

MURRELLS INLET SOUTH CAROLINA
HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1D HOUR 10

FIGURE 26

VELOCITY SCALE

0 2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA

HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1E HOUR 7

FIGURE 27

VELOCITY SCALE

2 0 2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE
MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA
HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1E HOUR 10

FIGURE 28

VELOCITY SCALE

2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA

HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1F HOUR 7

FIGURE 29

VELOCITY SCALE

TEST CONDITIONS

MODEL MEAN TIDE RANGE 4.8 FEET

MURRELLS INLET SOUTH CAROLINA
HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1F HOUR 10

FIGURE 30

VELOCITY SCALE

2 0 2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA

HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1G HOUR 7

FIGURE 31

VELOCITY SCALE

2 0 2 4 6 8

TEST CONDITIONS

ONLY M2 CONSTITUENT USED AS MODEL OCEAN TIDE

MODEL OCEAN TIDE RANGE = 4.8 FEET

MURRELLS INLET, SOUTH CAROLINA
HYDRAULIC MODEL STUDY

SURFACE CURRENT PATTERNS

PLAN 1G HOUR 10

FIGURE 32

TH CAROLINA
STUDY
E CURRENT
TERNS
HOUR 7

FIGURE 33

UTH CAROLINA
DEL STUDY
ACE CURRENT
ATTERNS
HOUR 10

URE 34

Appendix A

Design Calculations

APPENDIX A

Design of Weir and Deflector Dike Stone

Methods outlined in Section 7.38 of the Shore Protection Manual were used in sizing the stone for the jetty weir section and the Deflector Dike stone. The specific weight of the stone was assumed to be 160 pounds per cubic foot for all calculations.

BY D. V. Stutts DATE Sept 1976 SUBJECT Rubble Wier Design SHEET NO. 1 OF 1
CHKD. BY DATE MURRELLS INLET, SC JOB NO.

STONE SIZE DETERMINATION;

Ref: Sec. 7.373 Volume II Shore Protection Manual

Assumptions;

1. The SWL is taken to be Just above The top of the wier section @ +3 MLW
2. Breaking wave condition will exist
3. During severe weather & wave action fillet areas fronting wier will erode to −5 ft MLW.
4. Rubble Armor sized for Maximum breaking wave taken as .78 d_s.
5. Stability Coefficient, K_D, of 3.5 used, taken from Table 7-6 Shore Protection Manual

using eq. 7-105

$$W = \frac{W_r H^3}{K_D (S_r - 1)^3 \cot\theta}$$

where;

$W_r = 160\ \#/ft^3$

$H = .78 \times 8' = 6.2\ ft$

$S_r = {}^{160}/_{64.4} = 2.5$

$\cot\theta = 2$

$K_D = 3.5$

$$W = \frac{160\,(6.2)^3}{3.5(1.5)^3 2} = 1600\ lbs$$

BY D.V. Shutt DATE Sept 76 SUBJECT Murrels Inlet Deflector Dike Calculations SHEET NO. 1 OF 1

CHKD. BY ______ DATE ______ JOB NO. ______

REF: Shore Protection Manual CERC

Asumptions.

1. Maximum velocity measured in WES Model Study approx 3.5 ft/sec - not critical in sizing stone. - Design for wave cond.

2. Design wave selected a $H_{max} = 4.0 ft.$ Anticipated to be largest boat wave possible.

3. Stability Coefficient $K_D = 3.5$ from Table 7-6

4 Breaking wave condition using Eq 7-105

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot\alpha} = \frac{160 (4)^3}{3.5 (1.5)^3 2} = 433 \text{ lbs}$$

W = 433 lbs For Armor Layer

Approximate Spherical Diameter = D_{eq} = 1.6 ft

use 2 layer thickness for cover stone

weight range 325 lbs to 550 lbs.

Diameters range 1.45 ft to 1.73 ft.

APPENDIX B

STONE REQUIREMENTS

APPENDIX B

Stone Requirements

1. The gradation of the foundation blanket was selected in accordance with filter criteria in EM 1110-2-1601 in lieu of the gradation criteria in the Shore Protection Manual. The gradation limits required by the SPM would not function as a filter for the beach sands found at the project site. The EM gradations would contain smaller sizes that would act as a filter for the foundation sands.

2. The gradation limits of the toe protection were slightly increased over the SPM sizes to better resist wave action and ocean currents.

3. The core stone gradation sizes would be larger than those required by the SPM. However, the selected stone would be "quarry run"; therefore, special processing would not be required to produce the gradation specified. Since the primary purpose of the core stone is to serve as an economical substitute for armor stone, a quarry run gradation was selected. The top size of the core stone gradation was selected to be significantly large to prevent migration through the armor stone voids. The core stone and the toe protection were selected to have the same gradations to minimize costs and required stone sizes.

4. The armor stone sizes were selected according to SPM criteria. Normally, armor stone from granite quarries tends to be rectangular in shape and tends to interlock with adjacent stone such that voids would be small enough to prevent migration of the core stone.

END

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